DAM BREAK ANALYSIS USING HECRAS FOR NAGARJUNASAGAR DAM

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DAM BREAK ANALYSIS USING HECRAS FOR NAGARJUNASAGAR DAM

A thesis submitted in partial fulfillment of the requirements of the degree of

Master of Technology

in

Water Resource Engineering

by

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May 30, 2016

Supervisors Certificate

This is to certify that the thesis entitled, "**DAM BREAK ANALYSIS USING HECRAS FOR NAGARJUNASAGAR DAM"** *submitted by* **Mr. NARESH** *a part of requirements for the award of* **Master of Technology** *Degree in Civil Engineering at the National Institute of Technology, Rourkela is an authentic work carried out by him under our supervision and guidance.*

To the best of our knowledge, the matter embodied in the thesis has not been submitted to any other University/ Institute for the award of any Degree or Diploma.

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DECLARATION OF ORIGINALITY

I *Naresh Chinthu, Roll No: 214CE4085* hereby declare that this thesis entitled ***"**Dam Break Analysis using Hecras for Nagarjunasagar Dam"* presents my original work carried out as a student of NIT Rourkela and to the best of my knowledge, contains no material previously published or written by another person, nor any material presented by me for the award of any degree or diploma of NIT Rourkela or any other institution. Any contribution made to this research by others, with whom I have worked at NIT Rourkela or elsewhere, is explicitly acknowledged in the thesis. Works of other authors cited in this thesis have been duly acknowledged under the sections "Reference". I have also submitted my original research records to the scrutiny committee for evaluation of my thesis. I am fully aware that in case of any non-compliance detected in future, the Senate of NIT Rourkela may withdraw the degree awarded to me on the basis of the present thesis.

May 30, 2016 *Chinthu Naresh*

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Water Resource Engineering

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LIST OF ABBREVIATIONS

Particular	Description
1-D	One Dimensional
2-D	Two Dimensional
BT	Breach Time
BW	Breach Width
DB	Dam Break
DHI	Danish Hydraulic Institute
d/s	Downstream
DEM	Digital Elevation Model
DSL	Dead Storage Level
FRL	Full Reservoir Level
HD	Hydro Dynamic
HEC	Hydrologic Engineering Center
Km	Kilometer
LN	Lower Nagavali
m	Meter
Max.	Maximum
min.	Minutes
MWL	Maximum Water Level
NWS	National Weather Services
PMF	Probable Maximum Flood
Q	Discharge
Q-h	Discharge-Stage
s	Seconds
SCS	Soil Conservation Service
UK	United Kingdom

Abstract

In dam failure analysis provides the generating the flood inundation maps of Srisailam dam to Nagarjuna Sagar dam and its downstream. Flood inundation maps utilized to protect against the loss of life and property damage from maximum flood. The hydrologic engineering center's river analysis system (HEC-RAS) can be used in combined with HEC-GEORAS to develop a dam failure modes. For extract geometric information from digital terrain modes to used HEC-GEORAS and then imported in to HEC-RAS used for the unsteady flow simulation of dam break is performed and results are mapped in the ARC-GIS. In this thesis inundation mapping of water surface profiles provides a preliminary assessment of the flood hazard. The process of gathering data and preparing data and analysis of unsteady flow model in HEC-RAS, entry of dam breach parameters, performing the failure analysis and flood mapping in ARC-GIS is discussed.

This Thesis mainly provides an overview of the methods used to predict the breach outflow hydrographs with a detailed case study of hypothetical breach failure of dam "Nagarjunasagar dam" using HECRAS software. This Dam breaks are analyzed for failure with comparison of the hydrographs at different downstream locations by changing its breach parameter using HECRAS. The parameters describing a breach are typically taken to be the breach depth, width, side slope and breach formation time. Wahl (1998) and Wahl (2004) and Froehlich (2008) have found them to be very significant, especially the time parameter.

Chapter 1

Introduction

1.1 Background

There are more than 3300 dames listed in national inventory dam (NID) for the India. According to the Indian geological survey more than 150 of the dams are categorized as major dams. Major dams contain height of dam is 50 feet or more than 50feet, dam with normal storage capacity 5000 acre – feet and maximum storage capacity 25000 acre-feet or more than it. If these dams fail then places property and human life at risk. In fact, almost 170 dams would fails then result is loss of human life, property damage, and environmental damage.

As we know climate is continuously changing and which has introvduced unvcertainty in flow within the life span of dams. Many dams previously considered safe are now exhibit uncertainty in maximum flows which cause overtopping during high flood events leading to safety concerns. If a dam fails, loss of life and economic damage are direct consequences of such an event, depending on the magnitude of water depth and velocity, warning time, and presence of population at the time of the event. Early warning is crucial for saving lives in flood prone areas. The construction of dams leads people to believe that the floods are fully controlled, and therefore an increased urban and industrial development in the floodplains usually takes place. Hence, if the structure fails, the damage caused by flooding might be much greater than it would have been without the presence of it. Having the historical failures of structures in mind as discussed above, one might pose the question what can be done in order to reduce the risk posed from a dam failure event.

Dams are hydraulic structures of fairly impervious material built across a river or stream to create a reservoir on its upstream side for impounding water and utilize the flow of water for human purposes. Dams can be classified in number of ways but most usual ways of classification are based on function, structure and design. The classification based on structure and designs includes, concrete gravity, earth, rock fill, arch, buttress, steel, timber, and rubber dams while dams classified base on function are storage, detention, debris and coffer dams. The construction of dams in rivers can provide considerable benefits such as the supply of drinking and irrigation water, urban and industrial water supply, navigation as well as the generation of electric power and flood protection. However, the consequences which would result in the event of their failure could be catastrophic. They vary dramatically depending on the extent of inundation of area, population at risk and warning time duration.

Dam break may be summarized as the partial or catastrophic failure of a dam leading to the uncontrolled release of water. Flood can be defined as the occurrence of a very large amount of water in a very short time at a particular region caused mainly by heavy rain, melting of snow and dam failure. Dam failure results from both external force and internal erosion (Cederwall, 2005). Dam failure may arise due to different factors ranging from seepage, piping (internal erosion), overtopping due to insufficient spillway capacity and insufficient free board, extreme storm, foundation failure, earthquake, landslide, equipment malfunction and structure damage. The effect of such a disaster can be mitigated to a great extent if the resultant magnitude of flood peak and its time of arrival at different locations downstream of the dam can be estimated thus facilitating planning of the emergency measures.

Dam failure warrants dam break modeling which assesses the flood hydrograph of discharge from the dam breach due to propagation of flood waves along with their time of occurrence. A dam break may result in a flood wave up to tens of meters deep traveling along a valley at quite high speeds. The impact of such a wave on developed areas can be sufficient to completely destroy infrastructure such as roads, railways, bridges and buildings. Dam failure can lead to inevitable loss of life if advance warning and evacuation is not possible. Additional features of such extreme flooding include movement of large amount of sediments and debris along with the risk of distributing pollutants from any sources such as chemical works or mines in the flood risk area. Though there have been great advancement in design methodologies, failure of dams and water retaining structure can still occur.

Disaster is a sudden adverse or unfortunate extreme event which causes great damage to human being as well as plants and animals (Joshi, 2008). The United Nations defined disaster as “a serious disruption of the functioning of a community or a society causing widespread human, material, economic and environmental losses which exceed the ability of the affected community or society to cope using its own resources". A disaster happens when a hazard impacts on the vulnerable population and causes damage, casualties and disruption. Disaster Management plan can be defined as the organization and management of resources and responsibilities for dealing with all humanitarian aspects of emergencies in particular preparedness, response, and recovery in order to lessen or mitigate the impact of disasters. According to Warfield (2008), disaster management aims to reduce or avoid the potential losses from hazards, assure prompt and appropriate assistance to victims of disaster, and achieve rapid and effective recovery. The three key stages of activities that are taken up within disaster management are:

1. Before a disaster (pre disaster):

   Measures taken under this stage are mitigation and preparedness activities.

2. During a disaster:

   Activities carried out under this stage are called emergency and response.After a disaster (post disaster):

   Activities carried out under this stage are called response and recovery.

USACE Hydrologic Engineering Center is (HEC) Research document 13 lists causes of failure as follows: 1.Earthquake, 2.Landslide, 3.Extreme storm, 4. Piping, 5.Equipment malfunction, 6.Structure damage, 7. Foundation failure, 8.Sabotage. But what if above mentioned cause of dam failure occurs, huge volume of water with high speed travel along a downstream valley. The high flood wave generated from dam break is sufficient to destroy the developed areas there infrastructure, roads, railways, bridges and more important if advance warning and evacuation were not done than with loss of life of people the disaster becomes more painful to the society. As no program for preventing failure can ever be certain so to mitigate the risk associated with dam break the pre analysis is carried out.

DAM	LOCATION	DAM FEATURES	FAILES DUE TO
Kaddam project dam	Adiladad, Andhrapradesh Built in: 1957-58	Gravity dam Height: 30.78m Width:3.28m Full storage: 1.366*10^8 cum Max flood: 1.47*10^4 cum/sc Free board:2.4m	➢ Overtopped by 46cm of water above crest ➢ Breach width 137.2m
Kaila dam	Kachch, Gujarat Built in 1952-55	Gravity dam Height: 23.08m Crest length: 213.36m Full storage: 13.98*10^6 cum Free board:1.83m	➢ Piping by 3.12m below the river bed
Kodaganar dam	Tamilnadu Built in :1977	Gravity dam Height: 27m Full storage: 12.3*10^6 cum Flood capacity:1275 cum/sc Free board:2.5m	➢ Overtopping ➢ Breach width 20 to 200m
Machhu dam	Rajkot ,Gujarat Built in:1972	Gravity dam Height :23m Crest length: 3742m Full storage: 1.1*10^8 cum Free board: 24m	➢ Overtopped by 61cm of water above crest
Nanak sagar dam	Punjab Built in : 1962	Gravity dam Full storage: 2.1*10^6 cum Max flood:9711 cum/sec	➢ Overtopped ➢ Breach width: 45.7m

Panshet dam	Maharashtra near to Pune Built in :1961	Gravity dam Height :51m Full storage: 2.7*10^6 cum Max flood: 4870 cum/sec	➢ Overtopped by 60cm over the crest elevation ➢ Breach width :74m
Tigva dam	Sank, Madhya Pradesh Built in: 1971	Gravity dam Height of dam:24m Free board:2.4m	➢ Overtopped by 85cm of water above crest ➢ Breach width: 400m ➢ Estimated overflow discharge: 850 cum/sec
Khadawash dam	Mutha, Maharashtra Built in :1864	Gravity dam Height of dam: 32m Free board: 2.74m Max flood: 2775 cum/sec Full storage: 2.78*10^3 cum	➢ Overtopped ➢ Breach width: 65m

Table 1.1: Failure of dams in India

1.2 General

Dam fail due to as below causes

- If capacity of dam reservoir is exceeds from maximum flood then overtopping occurred.
- Internal erosion is due to piping of soil in embankment dams.
- Improper maintenance, including failure to remove trees, repair internal seepage problems, or maintain gates, valves, and other operational components.
- Improper structural design of dam or use of improper materials for construction of dam.
- If upstream side dam fails in the same drainage basin then flood occurred in dam sit then it's failed.
- Landslides into reservoirs, which cause surges that result in overtopping.

- Earthquakes, which typically cause longitudinal cracks at the tops of the embankments, leading to structural failure.
- Deliberate acts of sabotage

1.3 About HECRAS software

HEC-RAS is a one dimensional steady flow hydraulic model designed to aid hydraulic engineers in channel flow analysis and floodplain determination. The results of the model can be applied in floodplain management and flood insurance studies. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking and multi-user network environment. The system is comprised of a geographical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The HEC-RAS system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation.

HECRAS software is design by us army corps of engineers. HECRAS is used for modeling of water flowing through systems of open channel flow and computation of water surface elevations. HEC-RAS finds particular commercial application in floodplain management. For unsteady flow analysis hecras depends on 1-D saint venants equation.

1-D saint venants equation.

All of these assumptions combined arrives at the 1-dimensional Saint-Venant equation in the *x*-direction:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial h}{\partial x} + g(S - S_f) = 0$$

$$(a) \quad (b) \quad (c) \quad (d) \quad (e)$$

where (a) is the local acceleration term, (b) is the convective acceleration term, (c) is the pressure gradient term, (d) is the gravity term, and (e) is the friction term .

1.4 About HEC- Georgas

HEC-Georgas is contains a set of procedures, tools, and utilities for processing geospatial data in ArcGIS using a graphical user interface (GUI). The interface allows the preparation of geometric data for import into HEC-RAS and processes simulation results exported from HEC-RAS. To create the import file, the user must have an existing digital terrain model (DTM) of the river system in the Arc Info TIN format. The user creates a series of line themes pertinent to developing geometric data for HEC-RAS. The themes created are the Stream Centerline, Flow Path Centerlines, Main Channel Banks and Cross Section Cut Lines referred to ras themes. Additional RAS Themes may be created/used to extract additional geometric data for import in HEC-RAS. These themes include Land Use, Levee Alignment, Ineffective Flow Areas, and Storage Areas. Water surface profile data and velocity data exported from HEC-RAS simulations may be processed by HEC-Georgas for GIS analysis for floodplain mapping, flood damage computations, ecosystem restoration, and flood warning response and preparedness.

1.5 Scope of Thesis

Developing the dam break model and risk assessments due to flood produced from the dam break models for already constructed dams and dikes is becoming a necessity for a variety of reasons such as decreasing human casualties and economic damage. In this thesis, instead of focusing on already built hydraulic structures, we propose the analysis on two proposed medium dams by prediction of outflow hydrograph due to dam breach and it’s routing through the downstream valley to get the maximum water elevation and discharge along with time of travel at different locations of the river. For carry out the analysis HECRAS. Dam Break Model is used for Nagarjunasagar dam. Model is used to Estimate the consequences of Dam Break for downstream areas in terms of water elevation, travel time of flood waves, flow velocity etc. that cope up with hazards caused by structural failure events by decreasing their consequences. We consider events, though not likely to happen in any given year, if occurring is extremely catastrophic and have enormous socio–economic impact.

Chapter 2

Literature Review

Johnson and Illes (1976) He observed failure of earthen dams, gravity dams, and arch dams. Mainly in earthen dams, he described that mostly these dams failed due to trapezoidal and triangular breaches.

Singh and Snorrason (1982) conclude his studied 20 of dam failures and his concludes that variation of breach width was varying from 2 to 5 times of the height of the dam. The time taken for the complete failure of dam was 0.25 to 1 hr. in his study of 20 dam failure mostly failed due to over topping failure. This overtopping depth generally 0.15 to 0.61m over the crest elevate on.

MacDonald and Langridge-Monopolis (1984) proposed they described about breach side slope and breach formation time, volume of the breach outflow and depth of water above the breach at the time of failure. They concluded from analysis of the 42 case studies the most cases breach slope is to be 1h: 2v and the breach shape was triangular or trapezoidal.

Singh and Snorrason (1984): compare the results of DAMBRK and HEC-1 for eight hypothetical breached dams. Singh applied peak outflows with varying breach parameter using both models. In his results shows for large reservoirs the change in breach width produces larger changes (35% - 87%) in peak outflow and for small reservoirs the change is smaller in peak outflow (6% - 50%)

Yi xion(1985)

Described the dam break in the aspects of theories and models. Break parameters prediction, the understanding of dam break mechanics, peak outflow prediction were shown as the essential for the dam break analysis, and eventually determined the loss of the damages.

Petra check and Sadler (1984):

Petra check and Sadler described the sensitivity of discharge, inundation flood elevations, and flood arrival time with the change in breach width and breach formation time. For locations near the dam, breach width and breach formation time can have a more influence. For locations of downstream from the dam, the timing of the flood peak wave can be changed by changes in breach formation time, but the peak discharge and inundation elevations are insensitive to changes in breach parameters.

Froehlich (1987) created non dimensional forecast the equations for estimating average breach width, average side-slope factor, and breach formation time. The predictions were based on characteristics of the dam, volume of reservoir, height of water above the breach bottom, breach height, width of the embankment at the dam crest and breach bottom, and coefficients that account for overtopping vs. non-overtopping failures and the presence or absence of a core wall. Froehlich also concluded that, all other factors being equal, breaches caused by overtopping are wider and erode laterally at a faster rate than breaches caused by other means.

De Wrachien, et al.,(1987)

Disintegration of an earth dam can be prepared by low or weak point on the peak or on the downstream face by overtopping or piping dynamic disintegration then width and depth of breach, increasing out flow and disintegration rate.

Wurbs (1987):

Wurbs observed the small and large reservoir then concluded that breach width and breach formation time is depends on reservoir size.. The importance of different parameters varies with reservoir size. In large reservoirs, the maximum discharge occurred then the breach reaches its maximum depth and width. Changes in reservoir head then changes in the breach formation period and breach width is occurred. In these cases, accurate forecast of breach geometry is most critical. For small reservoirs, there is outstanding change in reservoir elevation during the formation of the breach, and as a result, the peak outflow occurs before the breach has fully developed. For these cases, the breach formation rate is the crucial parameter.

Lukman, et al., (1988)

The selected soil and geological properties of the dam were conducted with a particular attention to the release of water from the reservoir as seepage or filling of the reservoir by silt from erosion. Hydrology and hydraulic data of the study area and spillway were obtained and analyzed. It was concluded the dam has high seepage and silting rate. The rapid hydraulic conductivity, trees on embankment and grasses in the reservoir could have lead to the failure of the dam.

Singh and Scarlatos (1988):

They observation or survey of 52 case studies they documented breach geometry characteristics and time of failure of dam tendencies. They found that the ratio of top and bottom breach widths, (*B top/B bottom),* ranged from 1.06 to 1.74, with an average value of 1.29 and standard deviation of 0.180. The ratio of the top breach height to dam width was widely covered. The breach side slopes were inclined 10-50° from vertical in most cases. Also, most failure times were less than 3 hours, and 50 percent of the failure times were less than 1.5 hours.

Von Thune and Gillette (1990) and Dewey and Gillette (1993):

They used the data from Froehlich (1987) and MacDonald and Langridge- Monopolies (1984) to develop information for estimating breach side slopes, breach width at mid height of dam from crest elevation, and time for fully dam failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1h:2v or 1h:3v may be more appropriate.

Tony L. Wahl (July 1998), "Prediction of Embankment Dam Breach Parameters" U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office, July 1998.

Xiong (2011)

Using HEC-RAS hydraulic model, considering three scenarios for the simulation of Probable Maximum Flood (PMF) conditions which are the "without dam", "dam break", and "without dam break" scenarios. Using mixed flow regime simulation, both upstream and downstream boundary conditions (inflow hydrograph and rating curve) and the gate opening height were identified

Sunil Kute , Sayali Kakad (2013)

In them research they studied of Godavari river flood modeling using HEC-RAS software. The model facilitates to locate the flood plain and flood mapping of critical locations at downstream of dam and its extent for effective flood reduction measures.

Salah Eddine TACHI (2013)

Utilization of Geographical Information System (GIS) methods in incorporation with water powered displaying can altogether lessen the time and the assets required to estimate potential dam break flood danger which can assume a significant part in developing both flood disaster management and land use arranging downstream of dams.

Rasif Razack (2013)

Describes the analysis of a dam break in the aspects of simulation and various parameters. The parameters and outflow forecast are mainly for the understanding mechanism of dam break, which is essential for the dam break analysis, and especially a long delay determine the flood in each river station for a specific interval. The information content in HEC-RAS input and output files along with coordinate time (t) is recreated in a import data model to promote model interface and take advantage of GIS spatial analysis and visualization capabilities which gives an animated effect .Here I model this based on a limited geometric data.

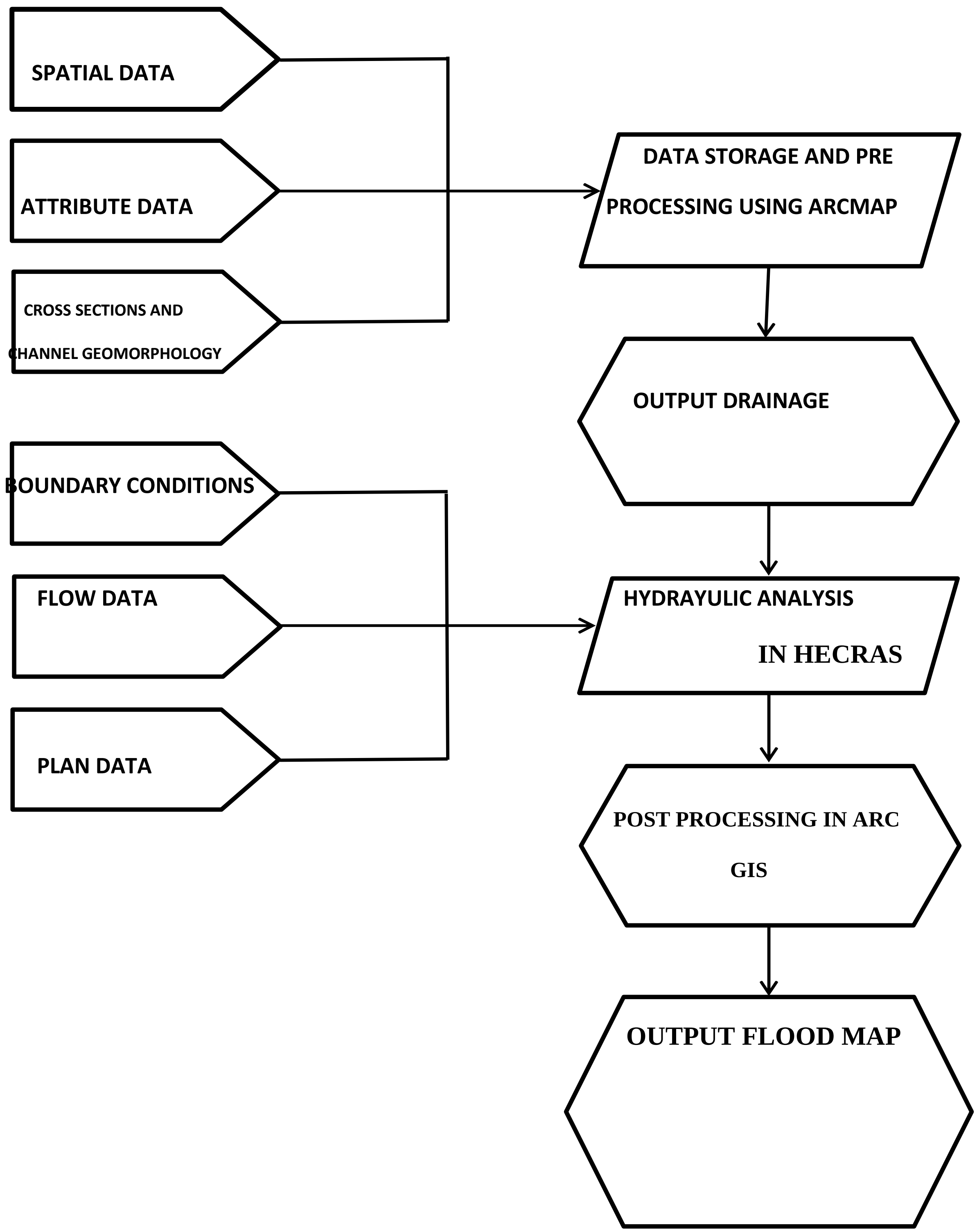
Purvang and Thakor,(2013)

They described to the process of studying a dam failure phenomenon and analyzing the resulting consequences at the downstream region. This generally deals with simulation of assumed failure for existing dams and analyzing the resulting consequences.

Chapter 3

Methodology and Simulation Procedure

3.1 Methodological frame work



3.2 Developing the HEC Georgas Import File

The main steps in developing the HEC Georgas Import File are as follows:

- Start a New Project
- Create RAS Layers
- Generate the HECGeoRas Import File

Start a New Project

Start a new project by opening a new Arc Map document. Then should be save the Arc Map project to before improve or develop any RAS Layers. This may require using the file folder to create and name a new file. The file to which the Arc Map project is stored it is to be the location for develop the RAS geodatabase and the location where the Georgas Import File is written.

Next, load the DEM in TIN/GRID format. To load the Terrain DEM, press the  (Add Layer) button on the Arc Map desktop. Select the TIN/GRID dataset from our browser file location and press OK. The DTM is added to the Arc map.

Create RAS Layers

The next step is to create the RAS Layers. This will be used for development and extraction of geometric data. In The RAS layers used to be develop the Stream Centerline, Banks, the Flow Path Centerlines and the Cross Section Cut Lines. A polyline layers of bridges/culverts, inline structures, lateral structures and a polygon layer for storage areas.

Create the shape file of our required feature class the Existing shape files are used for convert/import database fields to a feature class. The created layers must have a HydroID field and HydroId value. HydroID tool in ApUtilities is used for create the HydroID field and HydroID value.

Georgas geodatabase create is to design an empty feature class using the

(RAS Geometry | Create RAS Layers | *Feature Class)* menu items and copy and paste the features from our existing data set. Our Feature layers are created using basic ArcGIS drawing tools. The Georgas RAS Geometry menu describes the user through the data development procedure. The following section provides and develop for creating the RASLayers.

1. **Stream Centerline**

First should be creating The Stream Centerline layer. Select the (RAS Geometry | Create RAS Layers | Stream Centerline) menu item as shown in Figure

The Stream Centerline layer is drawing in the Map. If add the features to the Stream Centerline layer you will need to start an edit session on the feature class.

The Editor Tool bar used for Editing of features. The editing toolbar is loaded by selecting the (Tools |Customize) menu item and placing a checkbox next to the Editor Tool bar. The toolbar shown in below Figure

Fig 3.1: Editor Toolbar in ArcGIS

Select the (Editor | Start Editing) from menu item. Then select the geodatabase of our feature. Once select the any geodata base then you must develop the stream centerline layer. Lastly, select the Sketch tool and start drawing the river reaches one by one on the Arc map. River reaches should be drawn from upstream to downstream. Each river reach is represented by one line having a series of vertices. After creating the river network, save the edits in (Editor | Save Edits) and stop editing in (Editor | Stop Editing).

The Stream Centerline layer developed completely then each River and Reach has been assigned a name from select the  (Reach and River ID) tool. A Cross hairs will display as the cursor is moved over the arc map display. Select a River and Reach. Then create name to the river and reaches shown in below Figure. Previously specified river names and reach names are must be taken as different. If create the many reaches to one river then Reach names for the same river must be unique.

2. **Main Channel Banks**

If creating the Main Channel Banks layer in arc map then need not to define the bank station locations in HEC-RAS. Select the (RAS Geometry | Create RAS Layers | Bank Lines) menu item. Enter the layer name and press OK.

Start editing and draw the location of the channel banks. Draw the Separate lines for the left and right bank of the river. Bank lines from tributary rivers may overlap the bank lines of the main stem. After defining each bank line, save our edits in (Editor | Save Edits) and stop editing in(Editor | Stop Editing).

3. **Flow Path Centerlines**

Creating the Flow Path Centerlines layer for to find out distances between cross-sections of our river. If the flow path lines are not created in arc map then the distances between cross-sections should be added manually through the HEC-RAS interface.

Select the (RAS Geometry | Create RAS Layers | Flow Path Centerlines) menu item. Enter the layer name and press OK

If the Stream Centerline layer is already developed, the stream centerline is copied as the flow path for the main channel. Each flow path must be define with an identifier of *Left*, *Channel*, *Right*, corresponding to the left overbank, main channel, or right over bank. One by one, use the (Flow path) tool to define each flow path. After activating the Flow path tool, select each flow path with the cross-hairs cursor as shown in Figure 3-9 will appear allowing the user to select the correct flow path label from list. Assign the Flow Path Lines with Left, Channel, or Right

4. **Cross-Sectional Cut Lines**

Select the (RAS Geometry | Create RAS Layers | XS Cut Lines) menu item. Enter the layer name in the menu that appears and press OK.

Start editing and use the Sketch tool to draw the locations where cross-sectional data should be extracted from the terrain model. The cross-sectional cut line should be drawn from the left overbank to the right overbank, from facing downstream. Cross-sectional cut lines should be drawn perpendicular to the flow path lines. Cut lines must cross the main channel only once and no two cross sections may intersect Cross sections can be generated automatically at a giving interval and width using the (Construct XS Cut Lines) tool. This is the preferred method and should be used with caution because the lines are not generated following the guidelines necessary for modeling one-dimensional flow (i.e. cross sections could end up crossing each other and the main channel multiple times).

5. Inline Structures

Select the (RAS Geometry | Create RAS Layers | Inline Structures) menu item. Enter the layer name in the dialog that appears and press OK. Start editing and use the sketch tool to draw the locations where inline structure data should be extracted from the terrain model. Each inline structure cut line should be drawn from the left overbank to the right overbank, when facing downstream. You will also need to specify the top width and distance to the next upstream cross section in the Inline Structures attribute table.

6. Lateral Structures

Select the (RAS Geometry | Create RAS Layers | Lateral Structures) menu item. Enter the layer name in the dialog that appears and press OK .Start editing and use the sketch tool to draw the locations where lateral structure data should be extracted from the terrain model. Each lateral structure cut line should be drawn in the downstream direction. it will also need to specify the top width and distance to the upstream cross section just upstream in the Inline Structures attribute table.

7. Storage Areas

Select the (RAS Geometry | Create RAS Layers | Storage Areas) menu item. Enter the layer name in the dialog that appears and press OK. Start editing and use the sketch tool to draw polygons around areas that will act as floodplain storage.

8. Storage Area Connections

Select the (RAS Geometry | Create RAS Layers | Storage Area Connections) menu item. Enter the layer name in the dialog that appears and press OK.

Start editing and use the sketch tool to draw the locations where the storage area connection data (weir profile) should be extracted from the terrain model. Each storage area connection should be drawn from the left overbank to the right overbank, when facing downstream. it also need to attribute the connections with the nearest storage areas and weir’s top width.

3.3 Importing RAS Layers

The other option to creating a layer is to import an existing layer, and assign a HydroID value to each feature. The HydroID field is required for the Georgas tools to work. This is particularly useful for already have layers containing all the required fields in the specified format, though it is probably better to import the basic RAS Layers, assign attributes, and then create the associated 3D layers using the Georgas tools. If these layers are created in Georgas, the HydroID fields are automatically populated.

The HydroID field could be created and values assigned through the HydroID tool in the **ApUtilites** menu shown in Figure. The menu item will invoke a dialog that allows the user to specify a feature class (RAS Layer) and assign unique HydroIDs. If the HydroID field does not exist, the field will be created. Note that HydroID values must be assigned prior to attributing any of the feature classes. The HydoID provides the link from each feature to an attribute in an associated table. Assign Unique HydroIDs menu item is on the ApUtilites menu

3.4 Generating the RAS GIS Import File

After creating/editing each RAS Layer, select the RAS Geometry | Layer Setup menu item. The pre-processing layer setup dialog shown in Figure allows to select the RAS Layers used for data development and extraction. There are several tabs with dropdown lists. Click through each tab and select the corresponding data.

From the Required Surface tab, select the terrain data type: TIN or GRID. Use the drop down lists to select the Terrain TIN/GRID. From the Required Data tab, verify that the Stream Centerline layer and XS Cut Lines layer are selected. The XS Cut Line Profiles will be created by Georgas in a later step. From the Optional Layers tab, verify/select the layers you have created. Press the OK button when finished.

Next, select the RAS Geometry | Stream Centerline Attribute | Topology menu item. This process completes the centerline topology by populating from our menu. In addition, a table is also created to store nodes x, y, and z coordinates. These are used later to create the GIS import file. Select (RAS Geometry | Stream Centerline Attribute | Length/Stations) to assign length and station values to river features. Optionally, select RAS Geometry | Stream Centerline Attribute | Elevations to create 3D stream centerline layer from the 2D layer using elevations from the DTM. This step is not required – HEC-RAS does not use the elevation data extracted along the stream centerline!

The next step is to add geometric attributes to the Cross Section Cut Line layer. Select the items under the RAS Geometry | XS Cut Line Attributes menu one-by-one verifying the data that is appended to the XS Cut Line attribute table after each step. If an error message is invoked, fix your data set, and repeat the menu item. River and reach names, river station, bank station and downstream reach length information will be appended to each cross section cutline. To complete the cross-sectional data, station-elevation data needs to be extracted from the DTM. Select the (RAS Geometry | XS Cut Line Attributes | Elevations) menu item. This will create a 3D cross-sectional surface line layer from the cross-sectional cut lines.

We have an Inline Structures layer, select the (RAS Geometry | Inline Structures | River/Reach Names) to assign the River and Reach Names that the Inline Structures intersects. Select the (RAS Geometry | Inline Structures | Stationing) to assign station values to the Bridge/Culvert features. Select the (RAS Geometry | Inline Structures | Elevations) to create a 3D layer by extracting elevations from the DTM.

We have a Lateral Structures layer, select the (RAS Geometry | Lateral Structures | River/Reach Names) to assign the River and Reach Names that the Lateral Structure lies along. Select the (RAS Geometry | Lateral Structures | Stationing) to assign station values to the Bridge/Culvert features. Select the (RAS Geometry | Lateral Structures | Elevations) to create a 3D layer by extracting elevations from the DTM. We have storage areas, select the (RAS Geometry | Storage Areas | Elevation Range) to calculate the minimum and maximum elevation. Select the (RAS Geometry | Storage Areas | Elevation- Volume) Data to calculate elevation-volume relationship for each storage area of interest. Select the RAS Geometry | Storage Areas | TIN Point Extraction to extract all TIN points that fall within the storage area. (HEC-RAS does not currently use the points extracted within the Storage Area, therefore, skipping this step is recommended.) Lastly, select the (RAS Geometry | Extract GIS Data) menu item. This step writes the header information, river and reach information contained in the Stream Centerline layer, and cross-sectional information contained in the XS Cut Line Profiles layer to the RAS GIS Import File in the HEC-RAS spatial data format. Manning's n values, levee alignment data, ineffective flow data, blocked obstruction data, bridge/culvert data, inline structure data, lateral structure data, and storage data will be written, if available. This tool generates the RAS.GIS import file in two formats: one in the SDF format and the other in the XML format. The XML format is designed for future use. Note that this tool uses predefined XML and XSL files located under the folder in the HEC-Georgas install folder. These files are automatically installed, and must not be moved by the user. The tool expects to find these files at this location.

3.5 Running HEC-RAS

Create and save a new HEC-RAS project. From the Geometric Schematic choose the (File | Import Geometry Data | GIS Data) menu option. Select the *.RASImport.sdf* file to import. The Import Option dialog will appear as shown in Figure, though the dialog will be set to the Intro tab. Select the unit system to import the data into. Next, select the stream centerline by River and Reach name to import. Then select the cross sections to import by placing a check in the corresponding box. Select the properties to import for each cross section. When finished identifying the data for import press the (Finished – Import Data) button.

After importing the geometric data extracted from the GIS, completion of the hydraulic data will be necessary. Hydraulic data that may not be imported includes hydraulic structure data and storage areas. Flow data and the associated boundary conditions need to be supplied in developing of geometry in HECRAS.

Import and developing the geometry in HECRAS

After a new project is started, the user should open the Geometric Data Editor. Once the editor is opened, the user can import GIS/CADD data into HEC-RAS by selecting the Import Geometry Data - GIS Format option from the File menu of the Geometric Data window. When it is selected, a window will appear in which the user can select the file that contains the geometry data from the GIS.

Once the selects the file containing the GIS data, and then presses the OK button, a window will appear that will show what is available within the import file, and it will allow us to select what we want to import. The Import Options window will guide us through the process of importing all or part of the GIS import file. The initial tab of the Import Options dialog is the Intro tab, shown in Figure. HEC-RAS will read the import file and look for a “UNITS” tag. Based on the value associated with the tag, I will be offered the option to import the data in the current unit system or to convert the data from one unit system to another. If no unit system is found in the GIS file the import dialog will default to current RAS project units.

River Reach Stream Lines

The next tab on the import options is the River Reach Stream Lines This set of options allows to specify which river reaches to import, how to import the data, and what to name the river and reach. Import options for

Cross Section and Internal Boundary Nodes

The next tab on the Import Options window allows import cross sections and internal boundaries (bridges and inline structures). The Cross Sections and IB Nodes screen options should be drawn.

Storage Areas and Connections

The Storage Areas and Connections tab, shown in Figure, allows the specify storage areas and storage area connections to import and what name to import them withAfter making the selections of what to import, presses the Finished– Import Data button. The data will be imported and a schematic of the river system will show up in the Geometric Data window .Once the importing of the data is completed, it should save the geometric data by selecting Save Geometry Data As from the File menu of the Geometric Data window.

Manning's n Values

Several tables are also convenient for verifying and entering data. Manning's *n* value data may be entered using the (Tables | Manning n or k values) menu item. We use the manning's value is to 0.035 as per chow theory.

Entering and Editing Unsteady Flow Data

Once all of the geometric data are entered, the modeler can then enter any unsteady flow data that are required. To bring up the unsteady flow data editor, select Unsteady Flow Data from the Edit menu on the HEC-RAS main window. The Unsteady flow data editor should appear as shown in Figure.

It is required to enter boundary conditions at all of the external boundaries of the system, as well as any desired internal locations, and set the initial flow and storage area conditions at the beginning of the simulation.

Boundary conditions are entered by first selecting the Boundary Conditions tab from the Unsteady

Flow Data editor. River, Reach, and River Station locations of the external bounds of the system will automatically be entered into the table. Boundary conditions are entered by first selecting a cell in the table for a particular location, then selecting the boundary condition type that is desired at that location. Not all boundary condition types are available for use at all locations. The program will automatically gray-out the boundary condition types that are not relevant when the user highlights a particular location in the table. It can also add locations for entering internal boundary conditions. To add an additional boundary condition location, select either the Add RS button or the Add Storage Area button. The Add RS button allows users to enter additional river station locations for boundary conditions. The Add Storage Area button allows user to add storage area locations for insertion of a boundary condition.

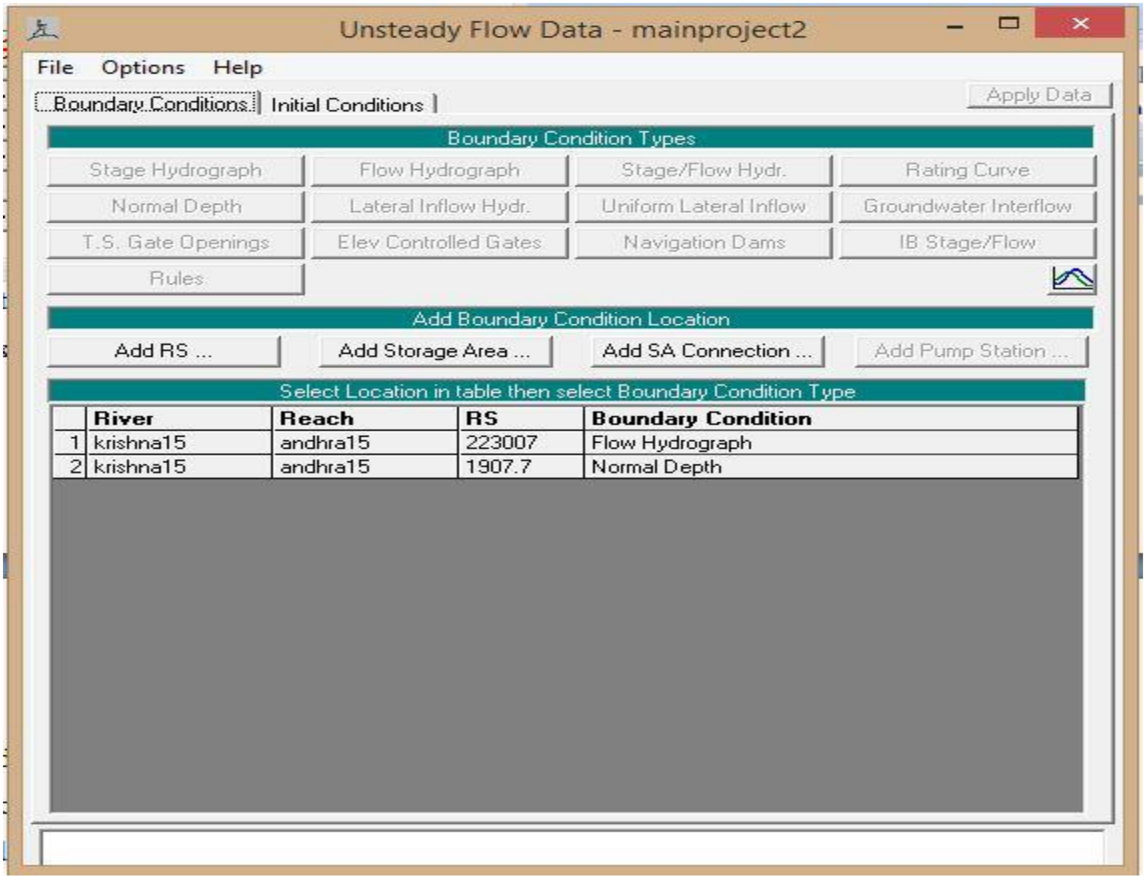


Figure 3.2 Editing of unsteady flow

Boundary Conditions

There are several different types of boundary conditions available to the requirement. The following is a short discussion of each type

Flow Hydrograph

Upstream Boundary:

For the Nagarjunasagar dam break model simulation, the Standard Probable Flood has been considered as a lateral inflow.

Time (hrs)	Discharge (cum/sec)	Time (hrs)	Discharge (cum/sec)
1	2548	27	68798
2	5086	28	142411
3	7604	29	220519
4	10092	30	302763
5	12540	31	388750
6	14939	32	478051
7	17279	33	570206
8	19551	34	664725
9	21745	35	761089
10	23854	36	858756
11	25869	37	957162
12	27782	38	1055721
13	29586	39	1153835
14	31272	40	1250890
15	32836	41	1346263
16	34270	42	1439326
17	35569	43	1529447
18	36727	44	1615995
19	37741	45	1698346
20	38606	46	1775879
21	39319	47	1847988
22	39877	48	1914083
23	40277	49	1973589
24	40519	50	2025957
25	40601		
26	40523		

A flow hydrograph can be used as either an upstream boundary or downstream boundary condition, but it is most commonly used as an upstream boundary condition. When the flow hydrograph button is pressed, the window shown in Figure will appear. As shown, it can either read the data from a HEC-DSS (HEC Data Storage System) file, or they can enter the hydrograph ordinates into a table.

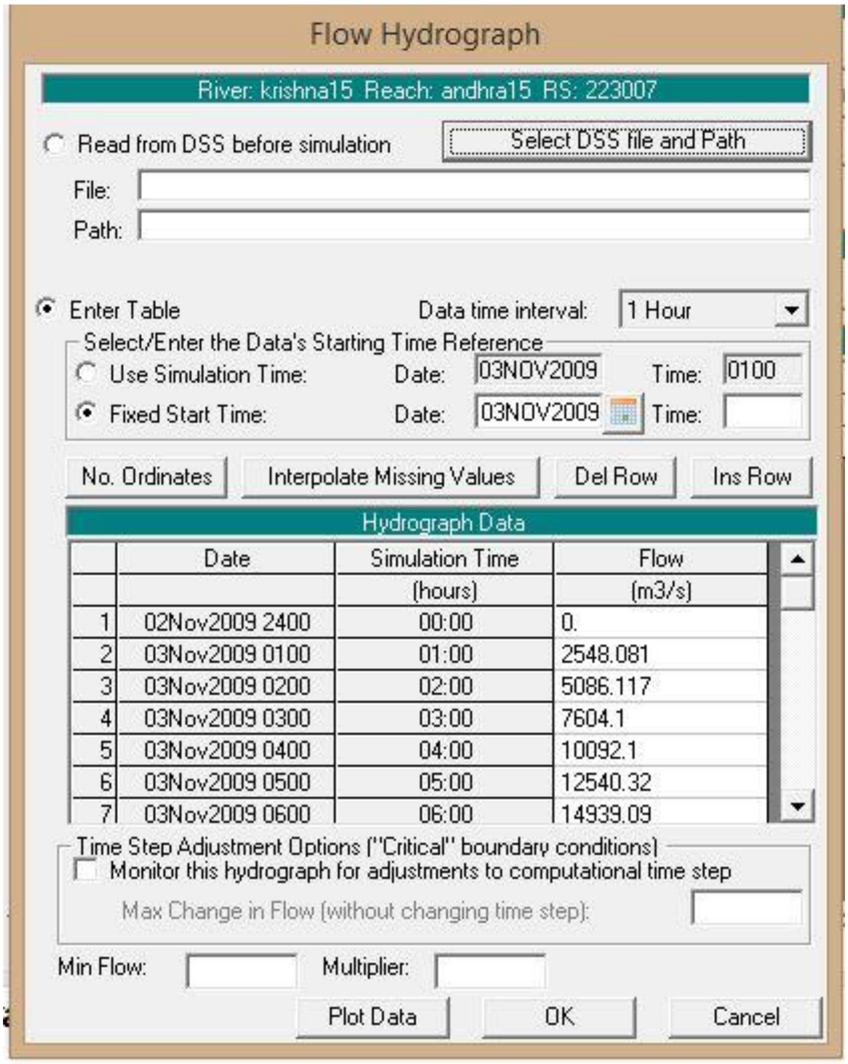


Figure 3.3: flood hydrograph

Normal Depth*:*

The Normal Depth option can only be used as a downstream boundary condition for an open-ended reach. This option uses Manning's equation to estimate a stage for each computed flow. To use this method it is required to enter a friction slope for the reach in the vicinity of the boundary condition. The slope of the water surface is often a good estimate of the friction slope. We take the normal depth is 0.055.

Initial Conditions

In addition to the boundary conditions, it must establish the initial conditions of the system at the beginning of the unsteady flow simulation. Initial conditions consist of flow and stage information at each of the cross sections, as well as elevations for any storage areas if defined in the system. Initial conditions are established from within the Unsteady Flow Data editor by selecting the Initial Conditions tab. After the Initial Conditions tab is selected, the Unsteady Flow Data editor will appear as shown in Figure.

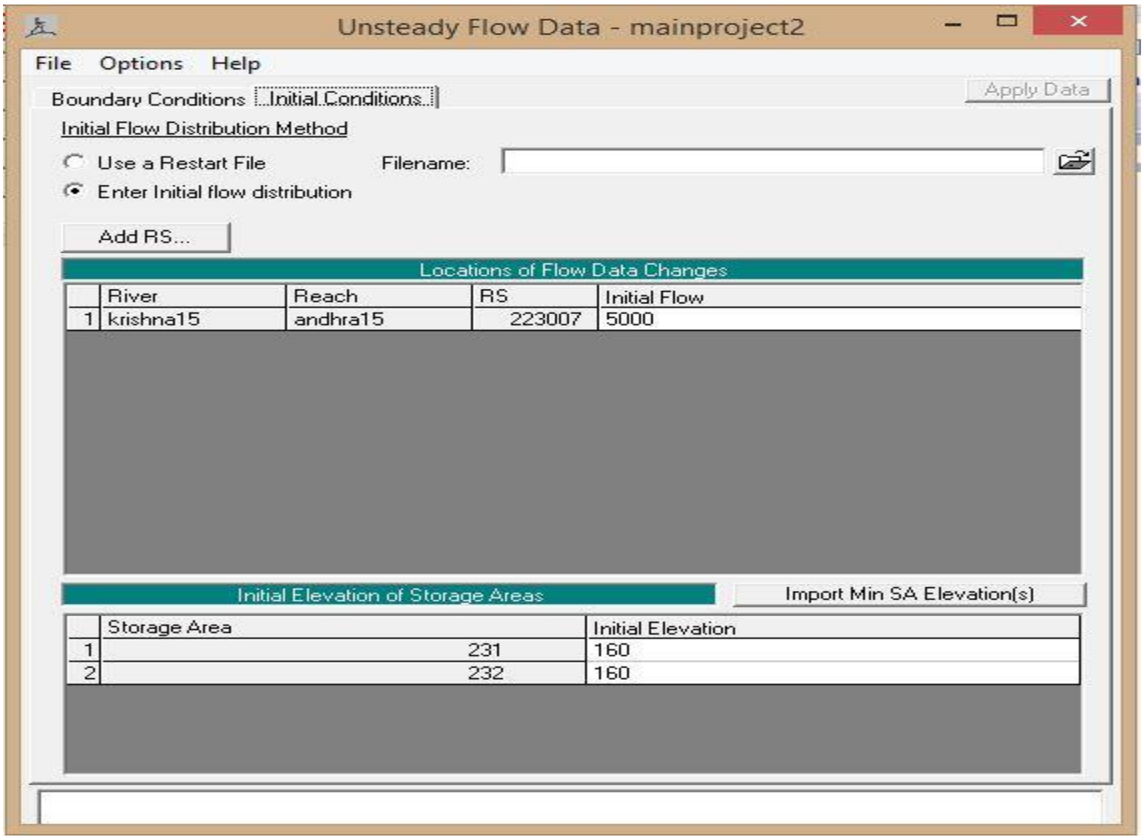


Figure 3.4: Entering of initial conditions

Calculation of breach parameters:

From the Formula of Froehlich (1995)

$$B_{avg} = 0.1803 k_o V_w^{0.32} h_b^{0.19}$$

$$T_f = 0.00254 V_w^{0.53} h_b^{-0.90}$$

B_{avg} = average breach width

V_w = reservoir volume (m^3)

h_b = breach height (m)

k_o = Constant multiplier

$$B_{avg} = 0.1803*1.4*(1.1565*10^{10})^{0.32}*(124)^{0.19}$$

$$B_{avg} = 1047.3m$$

$$T_f = 0.00254*(1.1565*10^{10})^{0.53}*(124)^{-0.9}$$

$$T_f = 7.09h$$

Entering of breach parameters

Enter the breach parameters From the Formula of Froehlich formula in the dialogue box as shown in below figure. In which enter the center station , final bottom elevation final bottom width, left and right side slopes of breach, breach weir coefficient, type of failure, starting water surface above the dam crest elevation.

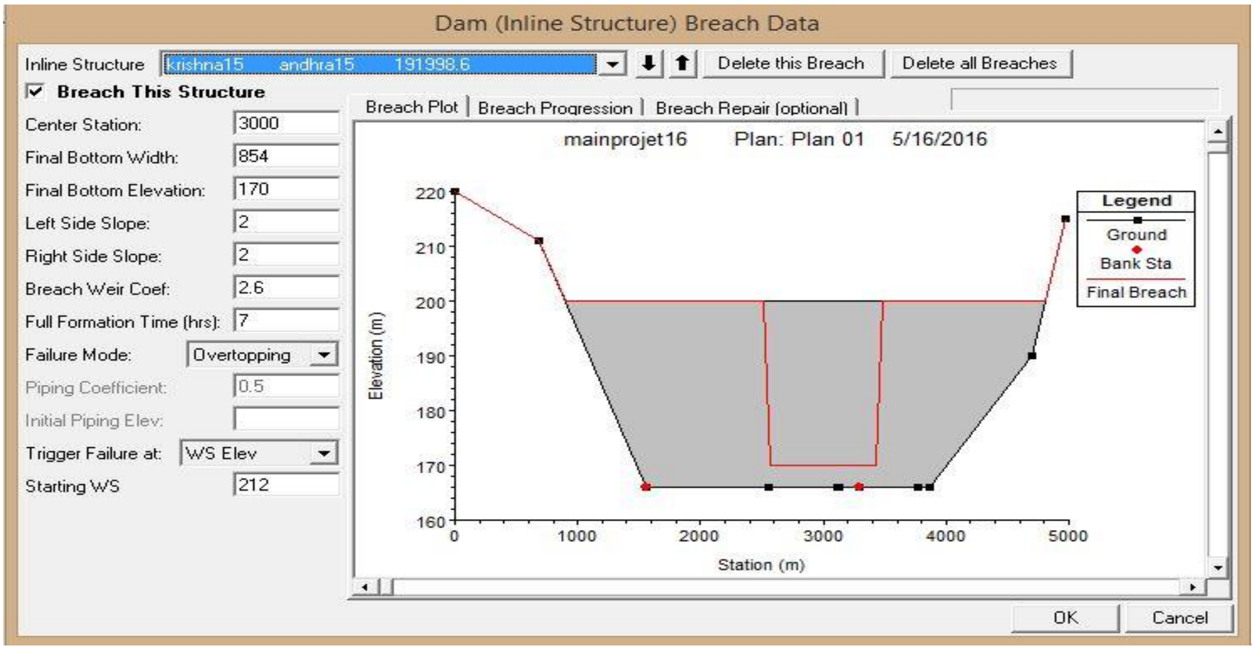


Figure 3.5: Entering breach parameters

Performing Unsteady Flow Calculations

Once all of the geometry and unsteady flow data have been entered, the user can begin performing the unsteady flow calculations. To run the simulation, go to the HEC-RAS main window and select Unsteady Flow Analysis from the Run menu.

Exporting the HEC-RAS Results

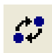
After unsteady flow simulation, HEC-RAS results can be exported for processing in the GIS by Georgas. Select the (File | Export GIS Data) menu option from the main RAS interface .The dialog shown in Figure will allow it to choose the file location to write the GIS information to and select the output options. Be sure to select the water surface profiles of interest. The GIS data will be written to the RAS GIS Export file (*.RASExport.sdf*).

3.5 Processing the RAS GIS Export File

The main steps in processing HEC-RAS results are as follows:

- Reading the RAS GIS Export File
- Processing RAS Results Data

Reading the RAS GIS Export File

The first step to importing HEC-RAS results into the GIS is to convert the SDF output data into an XML file, because the Georgas only use this format. Click the  (Convert RAS SDF to XML) button to execute this task. This tool initiates an external executable program, named SDF2XML.exe located under the folder, and dialog shown in Figure will appear. Select the RAS GIS Export File *(.RASExport.sdf)* in the. Click on the OK button convert this file to XML format.
The next step to importing HEC-RAS results into the GIS is to setup the necessary variables for post RAS analysis. Select the (RAS Mapping | Layer Setup) menu item. The dialog shown in Figure will appear to allow it to either start a new analysis or re-run an existing analysis. When you re-run an existing analysis, the variables input in the layer setup cannot be changed. For a new analysis, it need to specify a name for the analysis, RAS GIS Export File, terrain TIN/GRID, output directory, output geodatabase, dataset name, and a rasterization cell size.

The RAS GIS Export File is the XML export file generated in the previous step. Some post-processing results such as water surface tin and flood delineation grid will be saved into the output directory.

Import RAS data

After completion of layer setup import the RAS data from the RAS mapping menu in which click on import the RAS data then one dialogue box is created.

Inundation Results

After completion of import ras data then go to inundation mapping in which create the water surface generation. It is for creating water surface elevations in flow of our river. Then we can create the water surface TIN profile. In which to select Max Ws and press ok. For flood inundation maps to create the flood plain delineation using raster. This is occurred in menu of ras mapping in which go to inundation mapping then click on flood plain delineation using raster. Press ok. Then select the Max Ws and then ok. Then our flood map created.

Chapter 4

Study Area

4.1 Nagarjunasagar dam

Nagarjuna Sagar Dam was built across the Krishna River at Nagarjuna Sagar where the river is forming boundary between Nalgonda district of Telangana state and Guntur district of Andhra Pradesh state in India. The construction duration of the dam was between the years of 1955 and 1967. The dam created a water reservoir whose gross storage capacity is 11,472,000,000 cubic meters (4.051×10^{11} cu ft). The dam is 490 feet (150 m) tall from its deepest foundation and 0.99 miles (1.6 km) long with 26 flood gates which are 42 feet (13 m) wide and 45 feet (14 m) tall. Nagarjuna Sagar was the earliest in the series of large infrastructure projects termed as "modern temples" initiated for achieving the Green Revolution in India. It is also one of the earliest multi-purpose irrigation and hydro-electric projects in India.

The dam provides irrigation water to the Prakasam, Guntur, and Krishna, Khammam, West Godavari and Nalgonda districts along with hydro electricity generation. Nagarjuna Sagar dam is designed and constructed to utilize up to the last drop of water impounded in its reservoir of 405 TMC gross storage capacity which is the second biggest water reservoir in India. It is one of the earliest irrigation and hydro-electric projects in India. The dam provides irrigation water to the Nalgonda District, Prakasam District, Khammam District, and Guntur District. The proposal to construct a dam to use the excess waters of the Krishna river was put forward by the British rulers in 1903. Siddeswaram and Pulichintala were identified as the suitable locations for the reservoirs. The dam water was released by the then Prime Minister's daughter, Indira Gandhi in 1967.[5] The construction of the dam submerged an ancient Buddhist settlement, Nagarjunakonda, which was the capital of the Ikshvaku dynasty in the 1st and 2nd centuries, the successors of the Satavahanas in the Eastern Deccan. Excavations here had yielded 30 Buddhist monasteries, as well as art works and inscriptions of great historical importance. In advance of the reservoir's flooding, monuments were dug up and relocated. Some were moved to Nagarjuna's Hill, now an island in the middle of the reservoir. Others were moved to the mainland. The project benefited farmers in the districts of Guntur, Prakasam, Krishna, Nalgonda and Khammam. The right canal (a.k.a Jawahar canal) is 203 km long and irrigates 1.113 million acres (4,500 km²) of land. The left canal (a.k.a Lalbahadur Shastri canal) is 295 km long and irrigates 0.32 million acres (800 km²) of land in nalgonda and khammam districts of telangana region. The project transformed the economy of above districts. 52 villages were submersed in water and 24000 people were affected. The relocation of the people was completed by 2007.The hydroelectric plant has a power generation capacity of 815.6 MW with 8 units (1x110 MW+7x100.8 MW). First unit was commissioned on 7 March 1978 and 8th unit on 24 December 1985. The right canal plant has a power generation capacity of 90 MW with 3 units of 30 MW each. The left canal plant has a power generation capacity of 60 MW with 2 units of 30 MW each.

4.2 salient features of Nagarjunasagar dam

1. **Location**
   a) state : Andhra Pradesh
   b) District : Guntur
   c) River: Krishna
   d) Latitude & Longitude: 16°34′32″N 79°18′42″E
2. **Dam**
   a) Length of Masonry Dam : 1449.628 Mts.
   b) Height of Dam : 124.663 Mts.
   c) Length of Right Earth Dam : 853.440 Mts.
   d) Length of Left Earth Dam : 2560.320 Mts.
   e) Catchment Area at Dam Site : 215185 Sq.Kms.

   Maximum observed flood occurred on 03/08/2009 :42476 Cumecs

3. Reservoir :

a) Gross Capacity :408.240 TMC

b) Live Capacity as per minimum draw :202.470 TMC
Down elevation + 510 ft.

c) Water spread area : 285 Sq.Kms.

d) Dead Storage Elevation : +400.00 ft.

e) Full Reservoir Elevation : +590.00 ft.

4. spillway

a) Crest elevation of spillway : 166.42 m

Type of spillway gates : Radial

b) No of spillway gates : 26

c) Size of spillway gates : 13.71*13.41

d) Spillway capacity : 302 m^3/sec

Chapter 5

Result and Analysis

The most critical situation for the dam break is the condition when the reservoir is at full reservoir elevation and then peak of the most severe flood (PMF) impinges over the reservoir. As the spillway capacity is 42476 cumec which is similar to the peak Value of PMF. So it is obvious that spillway will discharge the peak of PMF without overtopping the dam crest elevation. For this study it is assumed that due to improper timing of gate opening at the time of PMF, the dam is just slightly overtopped by PMF and then dam is failed due to breaching. Since the dam is of earthen type the breach width and time of formation is calculated as From the Formula of Froehlich I'e 1074m, 7.03h respectively.

I selected the stations at a distance of 2km, 8km, 20km, 36km, 48km, 62km and 138km from the dam site. In these stations includes with villages and irrigation areas.

5.1 Flood analysis at various stations

The maximum discharge flows out from the breached dam is 105161 m^3/ s which is 2.48 times greater than the PMF. It is occurred at breach width 1074m and time of formation 7.03h at the location of 2km from the dam site. In same setup of breach width 1074m and time of formation time 7.03h at 8km from the dam is 104297 m^3/sec. It is 2.46times greater than the PMF. At the 20km the flood value is 97617m^3/sec which is 2.29 times greater than the PMF. At 36km it is 87705m^3/sec which is 2.06 times greater than the PMF. At the 48km it is 80148m^3/sec which is 1.88 times greater than the PMF. At the 62km it is 70723 m^3/sec which is 1.66 times greater than the PMF. At the station 138km from the dam the flood value is 59066 m^3/sec. The difference b/w flood at 2km and 138km is to be 46095 m^3/sec which is also greater than PMF.

Flood hydrograph at various station with breach width 1074m and breach formation time 7.03h with manning's value 0.035:

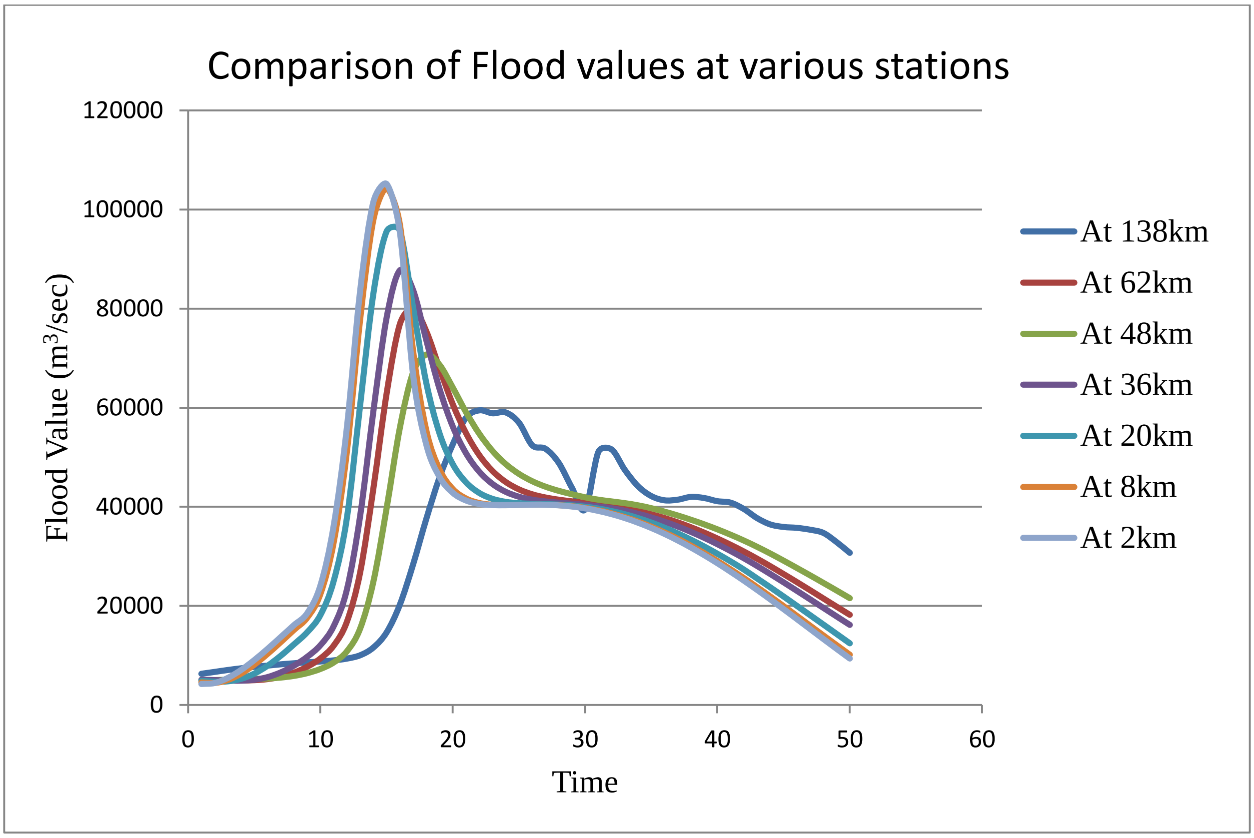


Figure 5.1 comparison of flood values at various stations

Flood value at 2km from the dam is 105161 m^3/sec and this flood value changes at station 138km is 28934 m^3/sec. This is half of the PMF value 42476 m^3/sec. The difference of first and last station flood values are 76227 m^3/sec which is 2.4 times greater than to our PMF value. Flood hazard occurred very critical at 2km from the dam

5.2 Water elevation at various stations

Water elevation scenario for five stations (2km, 32km, 48km, 62km, 138km) are explained. These stations because of in these stations includes with villages and irrigation areas.

Water elevation at various stations breach width 1074m and time of formation time 7.03h with manning's value 0.035:

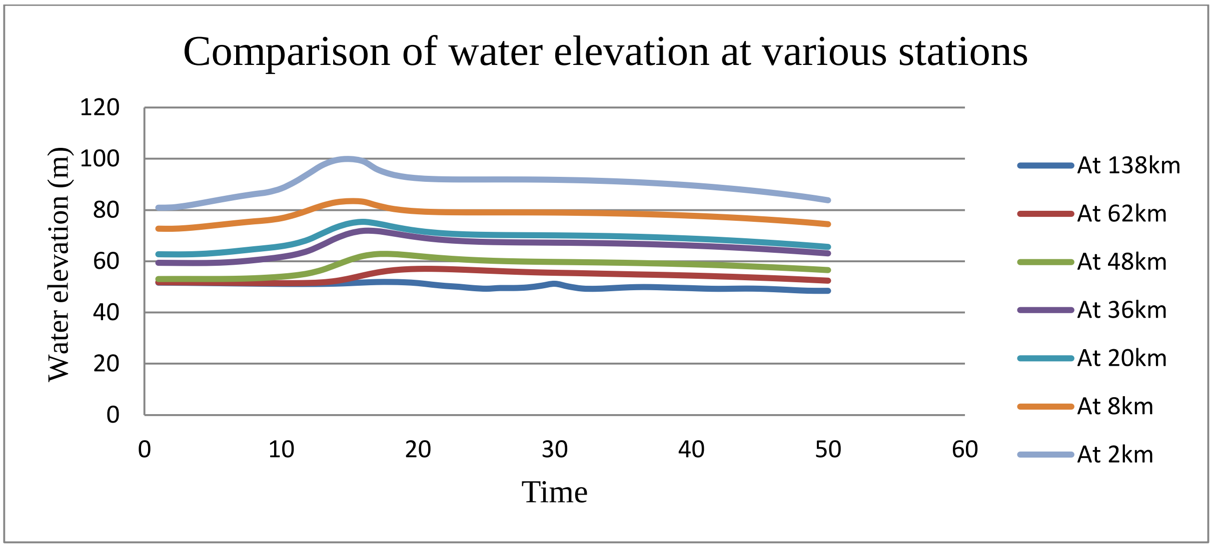


Figure 5.2 Comparison of water elevation at various stations

Water elevation maximum at station 2km is 99.71m it is decreases to 49m at last station of 138km from dam. Water elevation variation decreases as like as flood values of various stations.

5.3 Sensitivity analysis for various inputs to the model setup in terms of peak discharge and Water Elevations

As we know the selection of input parameters for the dam break model are very important to do the analysis. If we change the values of these input parameters to the model setup then what is the effect on discharge values and water elevations is analyzed and this analysis part is known as sensitivity analysis. So Input parameters which are considered for the sensitivity analysis are:

a) Breach time
b) Breach width
c) Manning's value

For the full study of Nagarjunasagar Dam break the results are obtained, analyzed and compared with different dam break scenarios as explained. Further the whole analysis is done on the different scenarios as explained bellow

Flood values of different breach widths and different time of formation with manning's value 0.035 at various locations:

At the station 2km from the dam site:

Table 5.1: Flood values of different breach widths at station 2km from dam

Breach width	7h	5h	3h	1h
1074	105161	121946	154853	206988
4*HD	104312	121376	152016	189121
3*HD	104046	119666	140962	164004
2*HD	92111	99963	108722	115837
1*HD	64687	66955	68442	68739

At station 2km is to be critical location. Because of flood values randomly increases any setup. in setup of formation time 7h and breach width 1074m flood value 105161 m^3/sec. it is decreases with respect to decreasing of breach width. But increases decreasing time of formation.

At the station 8km from the dam site:

Table 5.2: Flood values at various setup at station 2km from dam

Breach width	7h	5h	3h	1h
1074	104264	117517	147205	165077
4*HD	101678	116578	141977	150762
3*HD	100771	115821	129158	130276
2*HD	89078	94501	97285	98095
1*HD	61462	62457	64911	67907

At station 8km difference flood values of setup of breach width 1074 and breach time 7h to another setup of breach width 1074 and breach time 1h it is to be 60813 m^3/sec. The decreasing breach width then flood value also decreases as like as 104264 m^3/sec to 61462 m^3/sec.

At the station 20km from the dam site:

Table 5.3: Flood values at station 20km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	97617	108639	116446	118599
4*HD	96870	106304	108264	108526
3*HD	86981	98843	100323	100420
2*HD	79486	81367	82154	87416
1*HD	55385	57001	58179	59566

At the station 36km from the dam site:

Table 5.4: Flood values at station 36km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	87705	89506	90681	94945
4*HD	84879	88446	88501	89454
3*HD	83139	83813	84148	87706
2*HD	71835	71225	78725	79739
1*HD	54112	54161	56589	58189

At the station 48km form the dam site:

Table 5.5 flood values at station 48km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	80148	80767	87452	92001
4*HD	76753	78281	78438	81051

3*HD	76093	76247	76547	78205
2*HD	64768	65729	66931	59481
1*HD	50936	50958	58091	45645

5.3.1 Effect of breach formation time

Flood Hydrographs:

From above observation of different breach widths with different formation of time; at particular breach width of various locations if decrease the time of formation then flood value is increase. The maximum discharge of breach width 1074 and time of formation 7h is 105161 m^3/sec at the station 2km form the dam .At same station same breach width and time of formation 5h of maximum flood is 121946 m^3/sec which is 11.5 % greater than the peak value at formation time of 7h. if time of formation decreases to 3h then maximum flood value is 154853 m^3/sec which is 47 % greater than the flood value of time of formation 7h. and another setup of decreases the formation time is 1h then maximum flood is 206988 m^3/sec which is 96.82% greater than peck value of formation time of 7h. in analysis of dam break breach formation time is important criteria.

Flood hydrographs of various stations with same breach width of 1074m and different breach formation time:

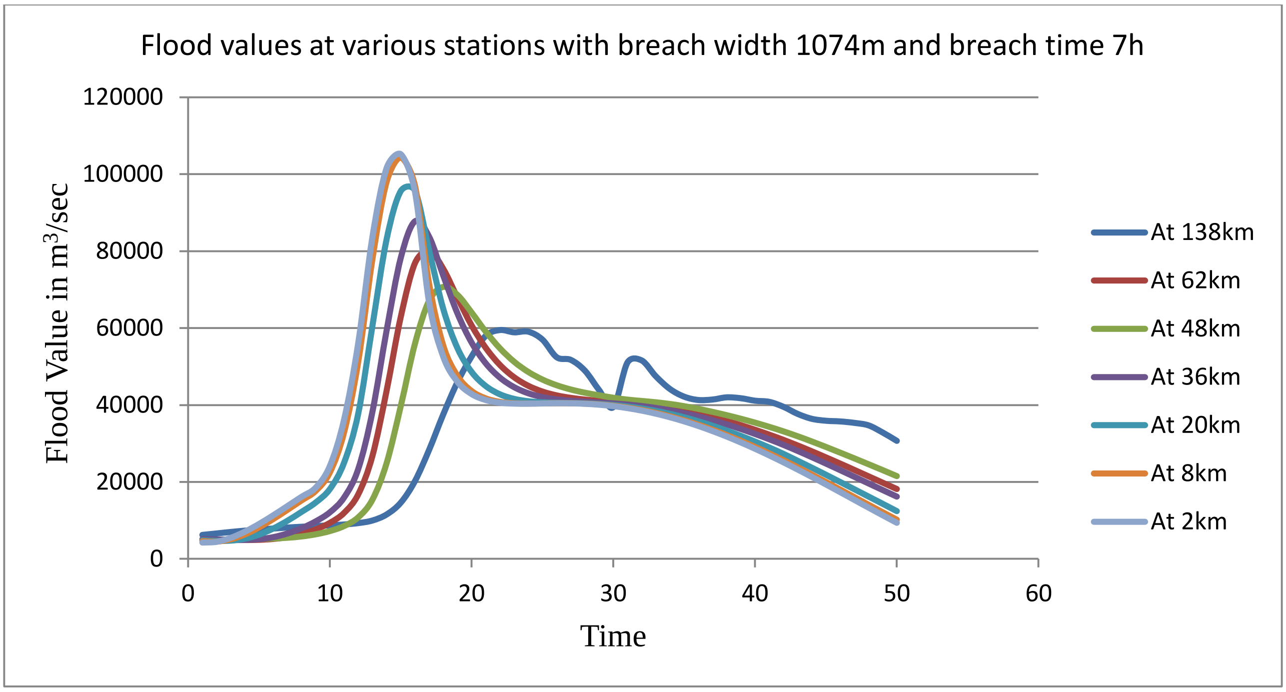


Figure 5.3: Flood values at various stations with breach width 1074m and breach time 7h

The maximum discharge of breach width 1074 and time of formation 7h is 105161 m^3/sec at the station 2km form the dam .At same station same breach width and time of formation 5h of maximum flood is 121946 m^3/sec which is 11.5 % greater than the peak value at formation time of 7h.

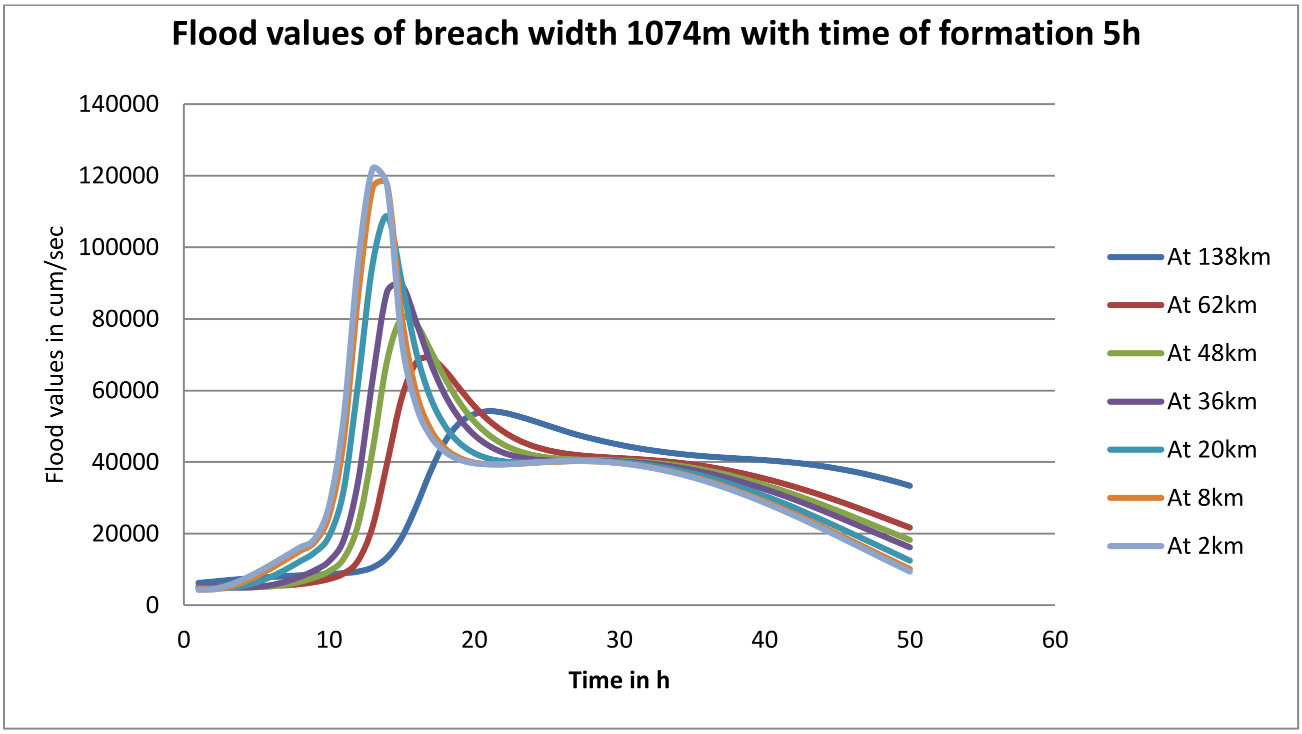


Figure 5.4 Flood values at various stations with breach width 1074m and breach time 5h

In this setup maximum flood 121946 m^3/sec is occurred at 2km location it is decreasing value at 138km is to be 48744 m^3/sec. But flood value occurred at last location is greater than our PMF value.

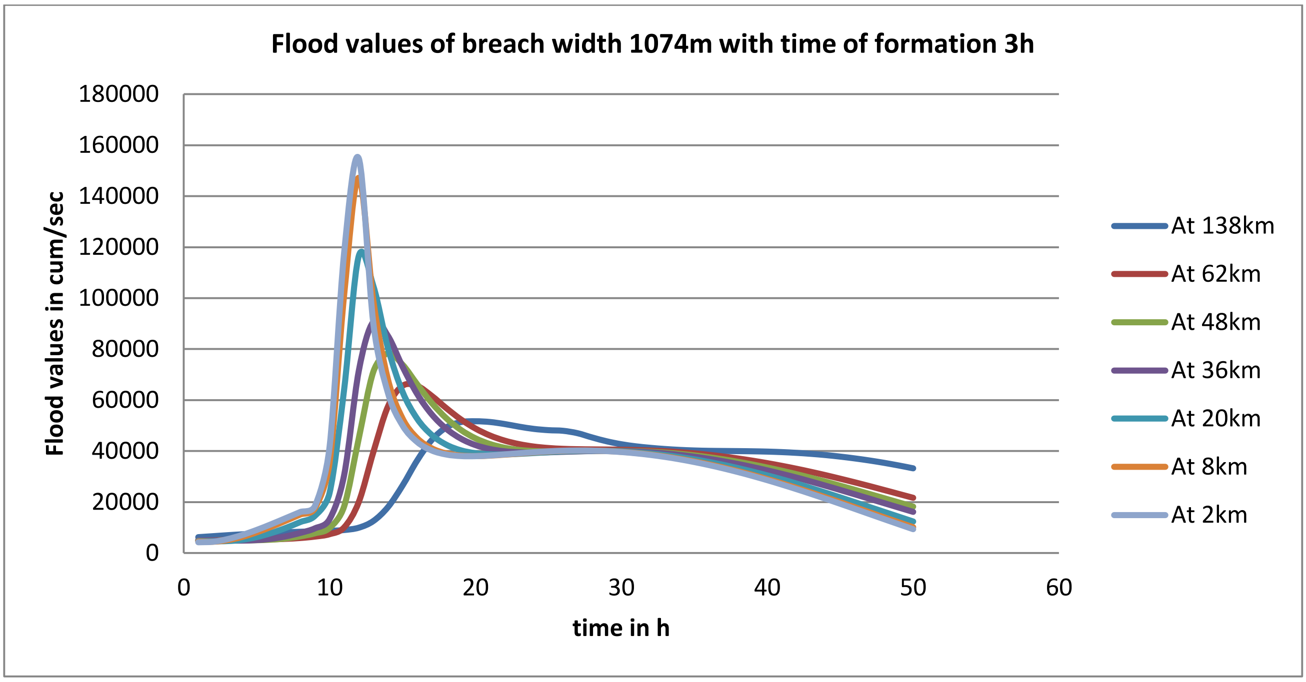


Figure 5.5 Flood values at various stations with breach width 1074m and breach time 3h

In this setup flood values are greater than before setup value this is critical setup than breach width 1074 and breach time 5h. The maximum flood value breach width 1074m and breach time 5h is 121946 m^3/sec. At breach width 1074m and breach time 3h it flood value is 154853 m^3/sec.

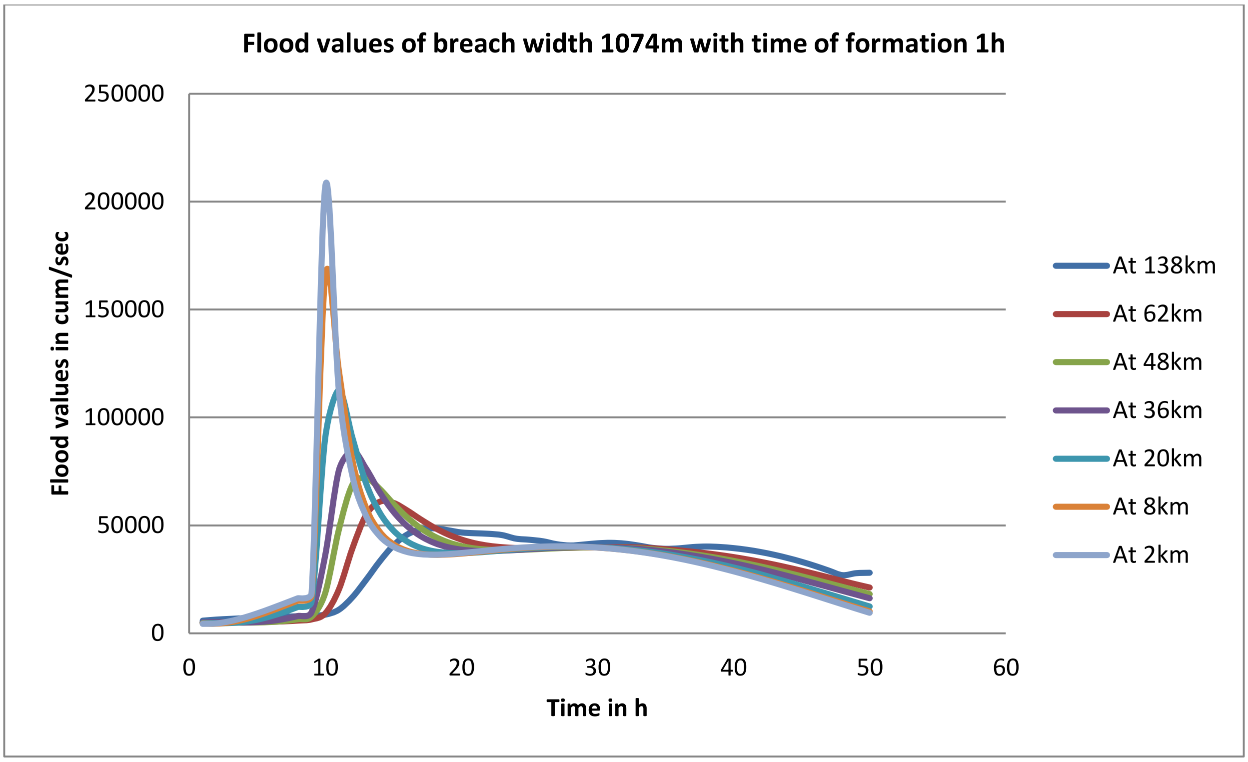


Figure 5.6 Flood values at various stations with breach width 1074m and breach time 1h

This setup is very critical than reaming setup. Because of flood value is occurred 206988 m^3/sec which is 5 times greater than our flood value. So if this setup is occurred then stations of 2km ,8km, 20km locations are very critical.

Water elevations:

Water elevations of different breach width and different time of formation at various stations:

At the station 2km from dam

Table 5.6: Water elevation at station 2km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	99.91	101.39	104.15	107.55
4*HD	99.89	101.32	103.85	106.33
3*HD	99.79	101.22	102.96	104.53
2*HD	98.48	99.25	99.99	100.32
1*HD	95.39	95.62	95.73	95.85

At the station 8km from dam:

Table 5.7 Water elevation at station 8km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	83.49	84.16	8513	85.26
4*HD	83.32	84.13	84.89	84.81
3*HD	83.41	83.94	84.32	84.64
2*HD	82.54	82.77	82.78	82.74
1*HD	80.8	80.96	80.79	81.13

At the station 20km from dam:

Table 5.8 Water elevation at station 20km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	75.32	75.6	76.9	79.94
4*HD	75.17	75.19	75.23	79.55
3*HD	74.69	74.72	74.78	75.87
2*HD	73.73	73.56	73.85	74.02
1*HD	71.45	71.22	71.49	71.62

At the station 36km from dam:

Table 5.9: Water elevation at station 36km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	71.91	72.1	72.92	74.44

4*HD	71.82	71.95	72.53	74.28
3*HD	71.59	7169	71.42	72.83
2*HD	70.63	70.48	71.11	71.81
1*HD	68.83	68.9	68.9	69.04

At the station 48km from dam:

Table 5.10 Water elevation at station 48km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	62.8	62.8	62.98	63.83
4*HD	62.57	62.52	62.95	63.68
3*HD	62.1	62.24	62.33	62.48
2*HD	61.92	61.61	61.76	61.89
1*HD	60.52	60.4	60.66	60.84

From above observation at a constant breach width of 1074m then decreases the breach formation time water elevation at various stations is to be increases. At the station of 2km from dam, in which water elevations occurred 99.71m at 7h of formation time. At 5h of formation time it is 101.39m, at 3h of formation time it is 104.15m, at 1h of formation time water elevation is 107.55m. Water elevations increase with decrease the formation time. It occurred because of in that setup increase maximum flood values so increase the water elevations.

Flood hydrographs of various stations with same breach width of 1074m and different breach formation time:

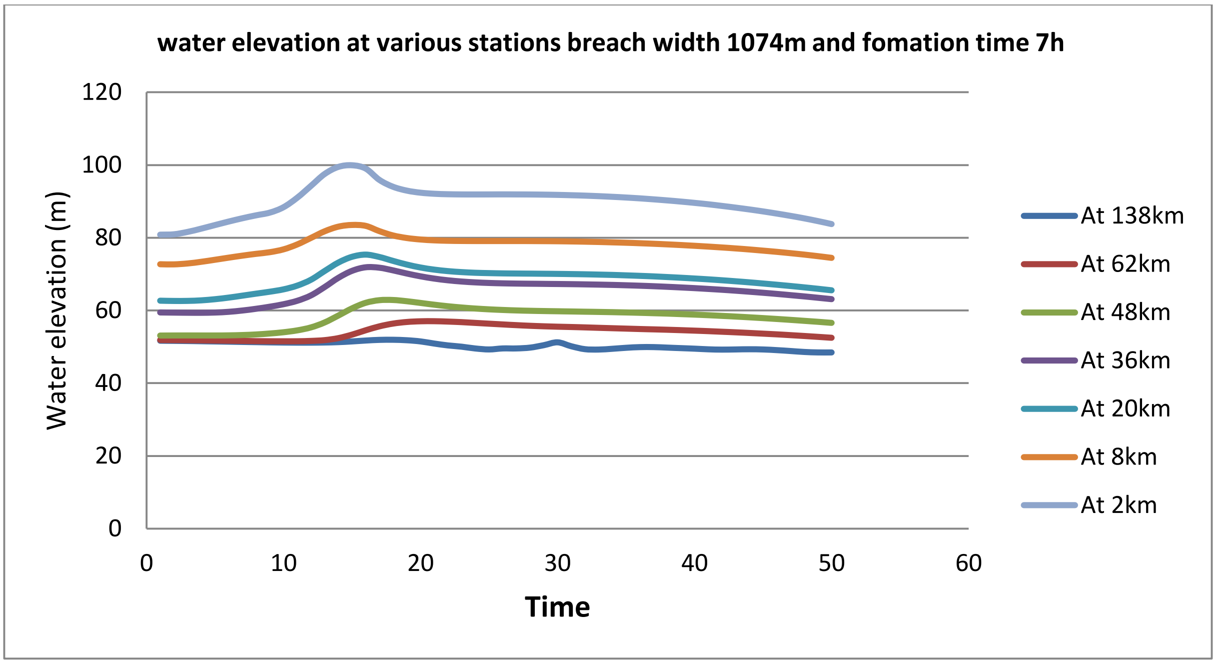


Figure 5.7 Water elevation at various stations with breach width 1074m and breach time 7h

Maximum water elevation occurred at 2km location is 99.71m it is decreases water elevation is 49m at 138km.

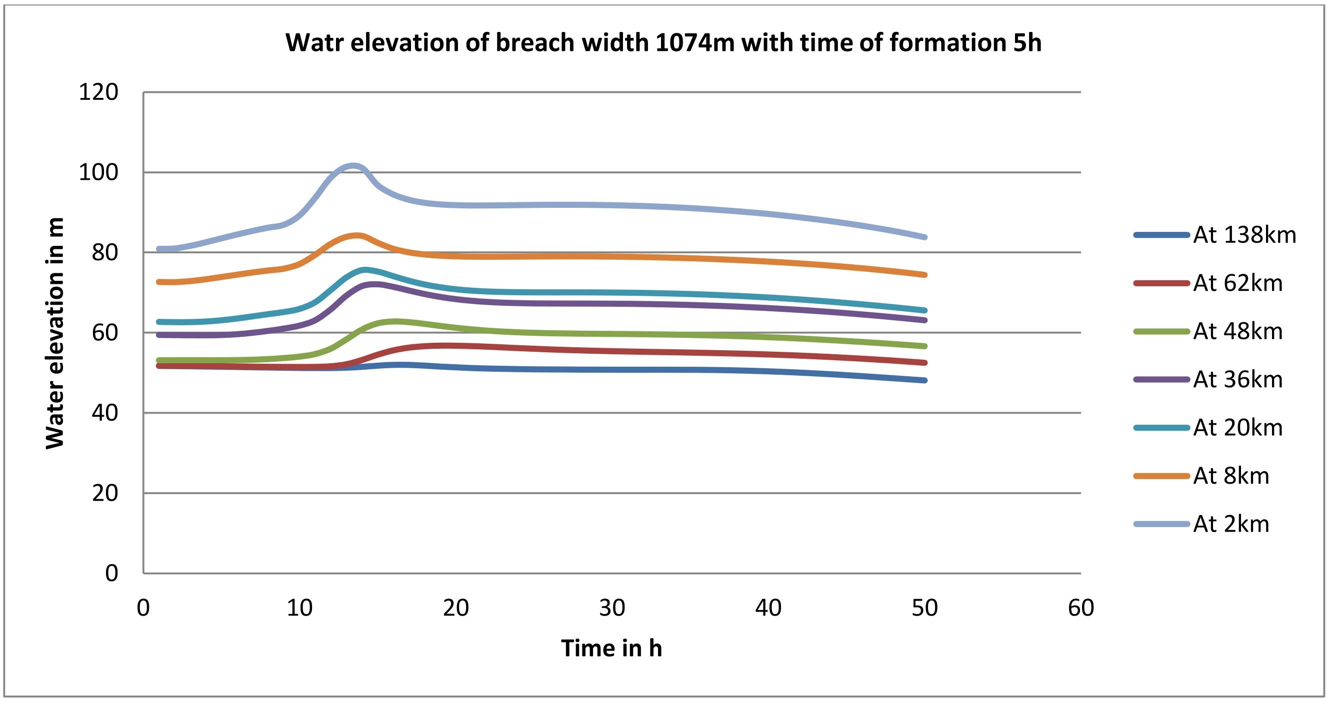


Figure 5.8 Water elevation at various stations with breach width 1074m and breach time 5h

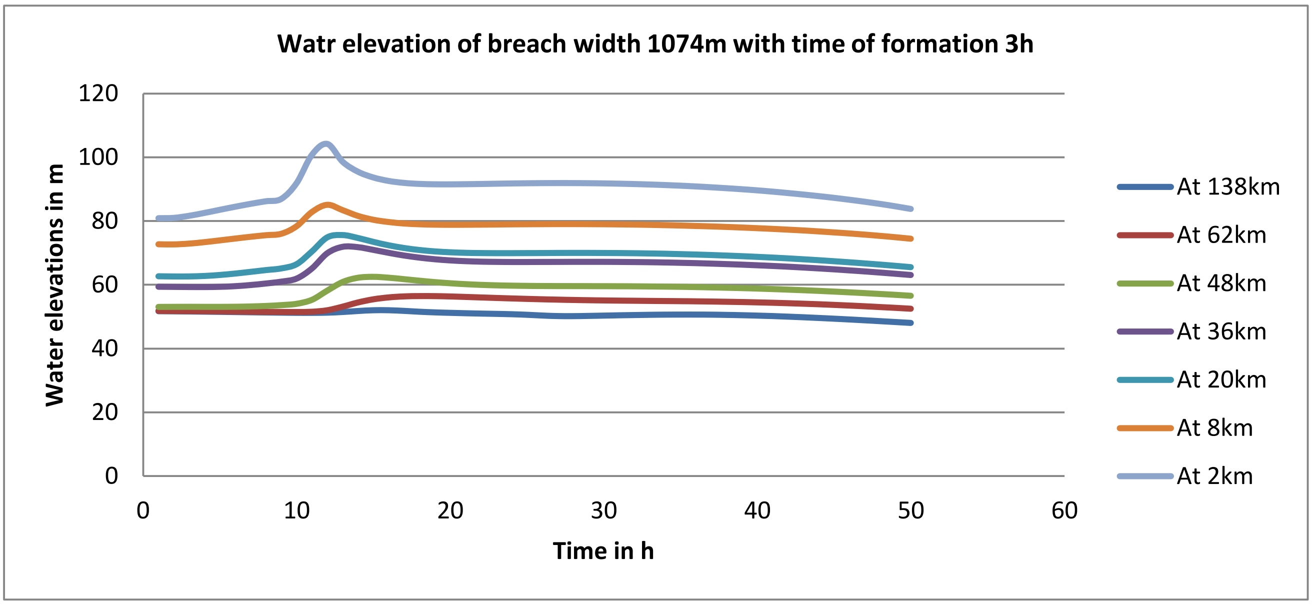


Figure 5.9: Water elevation at various stations with breach width 1074m and breach time 3h

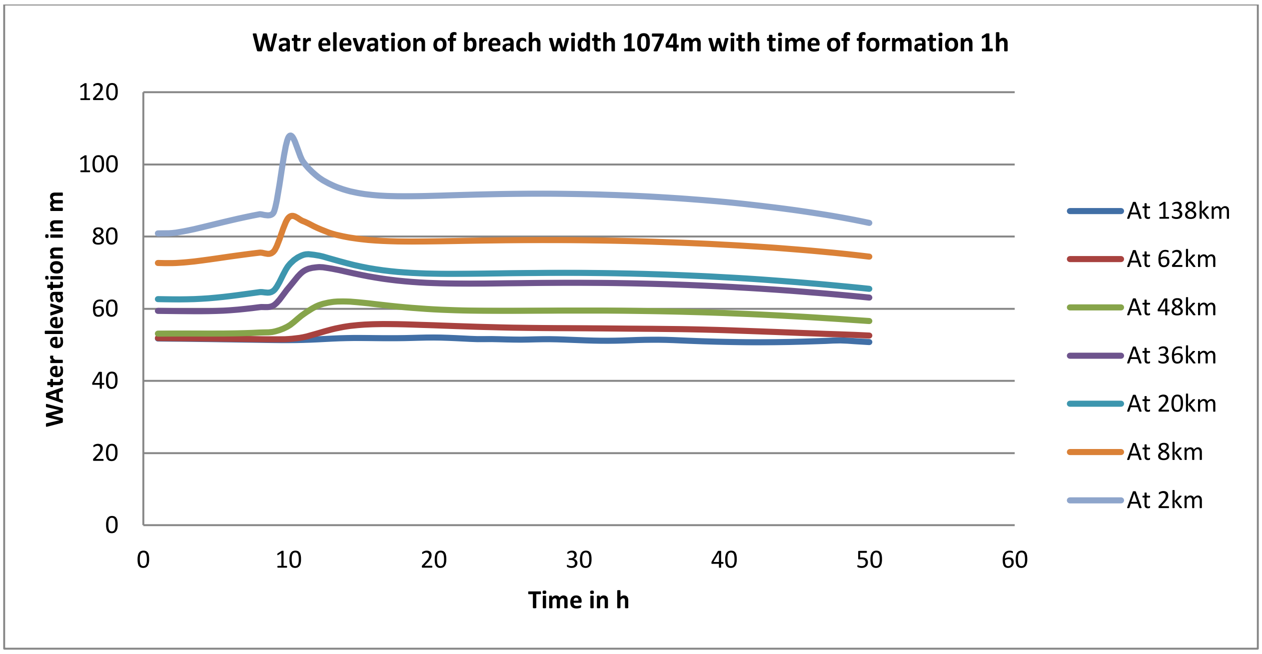


Figure 5.10 Water elevation at various stations with breach width 1074m and breach time 1h

In this setup of maximum water elevations are occurred because of flood values are maximum at this setup. Water elevation at station 2km is 107.55m it is critical water elevation.

5.3.2 Effect of breach width:

Flood values:

Table 5.11 Flood values at station 2kmkm with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	105161	121946	154853	206988
4*HD	104312	121376	152016	189121
3*HD	104046	119666	140962	164004
2*HD	92111	99963	108722	115837
1*HD	64687	66955	68442	68739

From the above the table at constant time of formation , Decreases breach width then maximum flood value is to be decreases .The maximum flood value at 7h time of formation and breach width 1074m is 105161m^3/sec. at breach width 4*HD it is 104312 m^3/sec which is decrease compare with breach width of 1074m. At breach width 4*HD to 3*HD then flood value decrease 104312 m^3/sec to 104046 m^3/sec. As similarly at breach width of 2*HD, 1*HD of flood values deceased as 92111 m^3/sec, 64687 m^3/sec. maximum flood of 1*HD breach width is compare with breach width of 1074m it is 62.5% less. . So, with the change of breach width there is slightly increase in peak discharge from the breach dam and almost same peak water elevation along the downstream location is observed.

Flood values at same time of formation (7h) and different breach width at various stations:

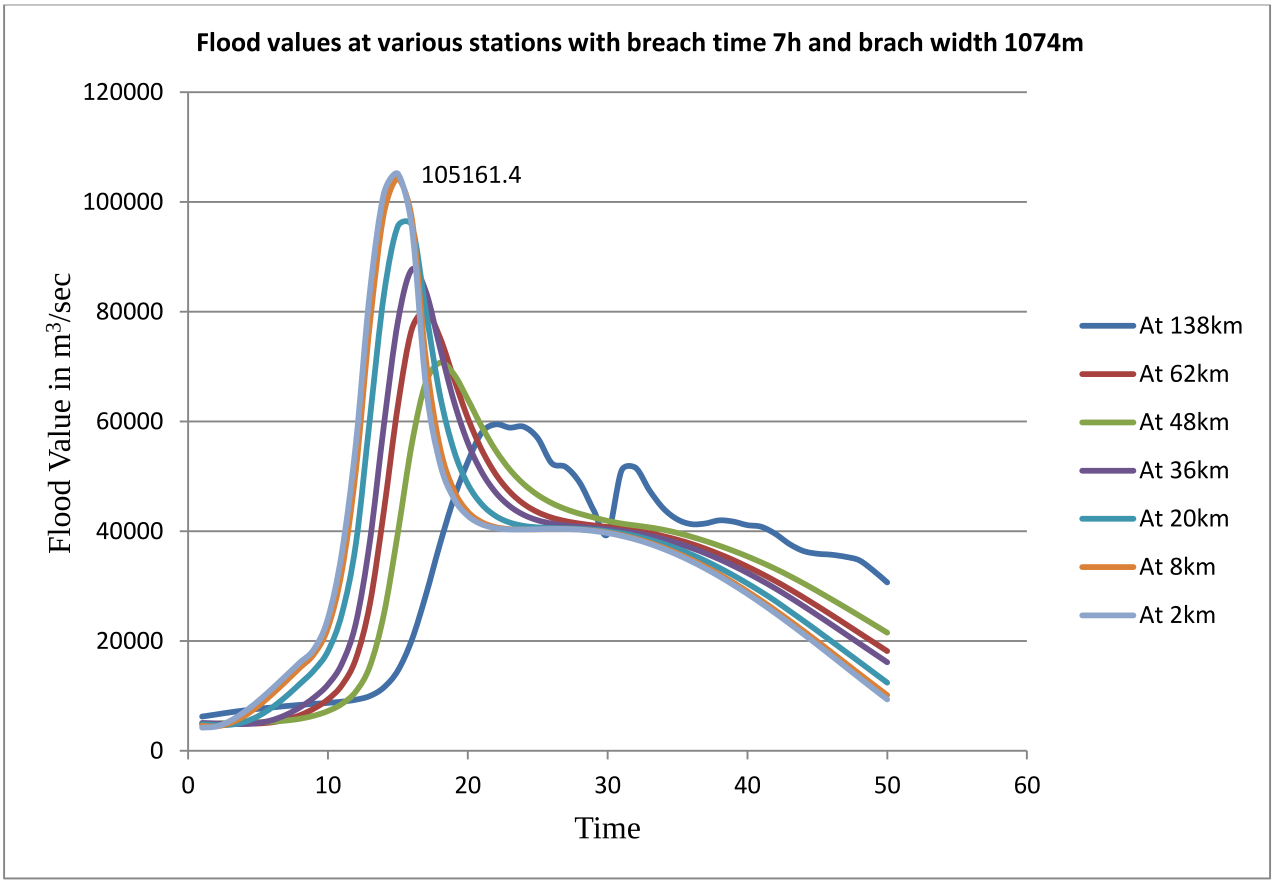


Figure 5.11: Flood values at various stations with breach width 1074m and breach time 7h

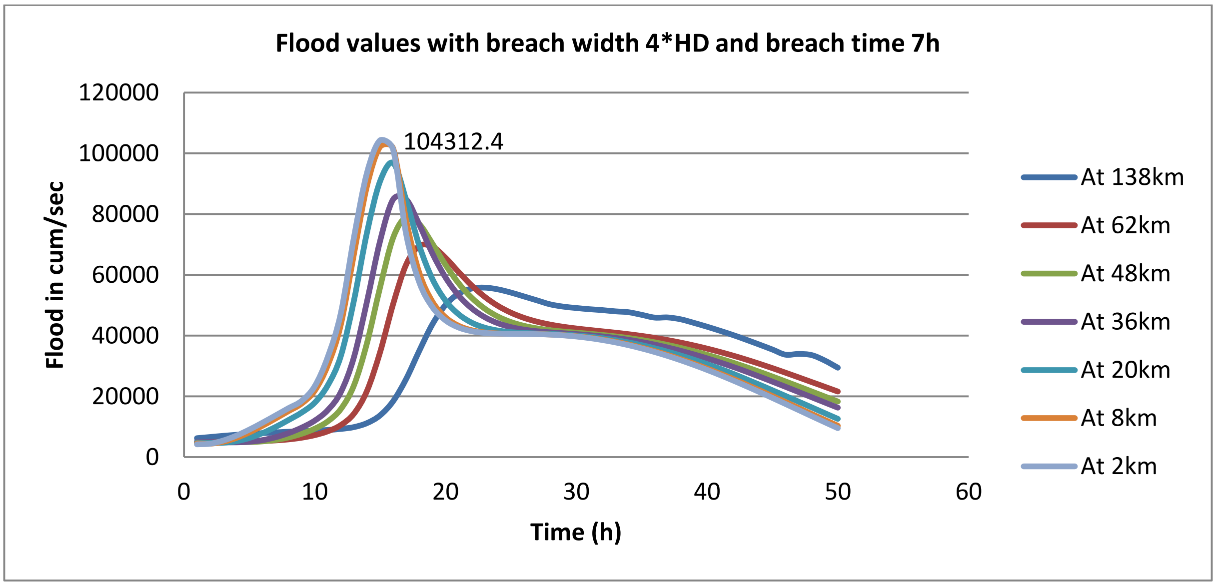


Figure 5.12 Flood values at various stations with breach width 4*HD and breach time 7h

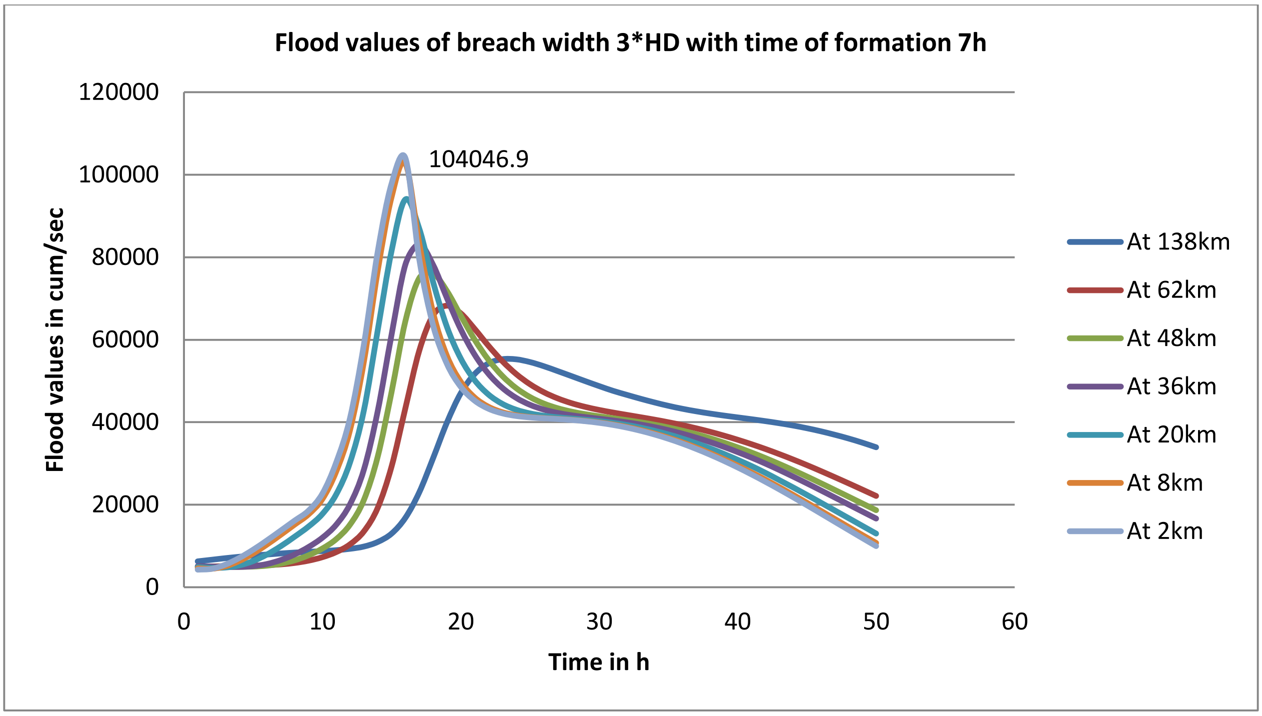


Figure 5.13 Flood values at various stations with breach width 3*HD and breach time 7h

Maximum flood value at this setup is 104046 m^3/sec at 2km location and maximum flood value at 138km location it is 31259 m^3/sec.

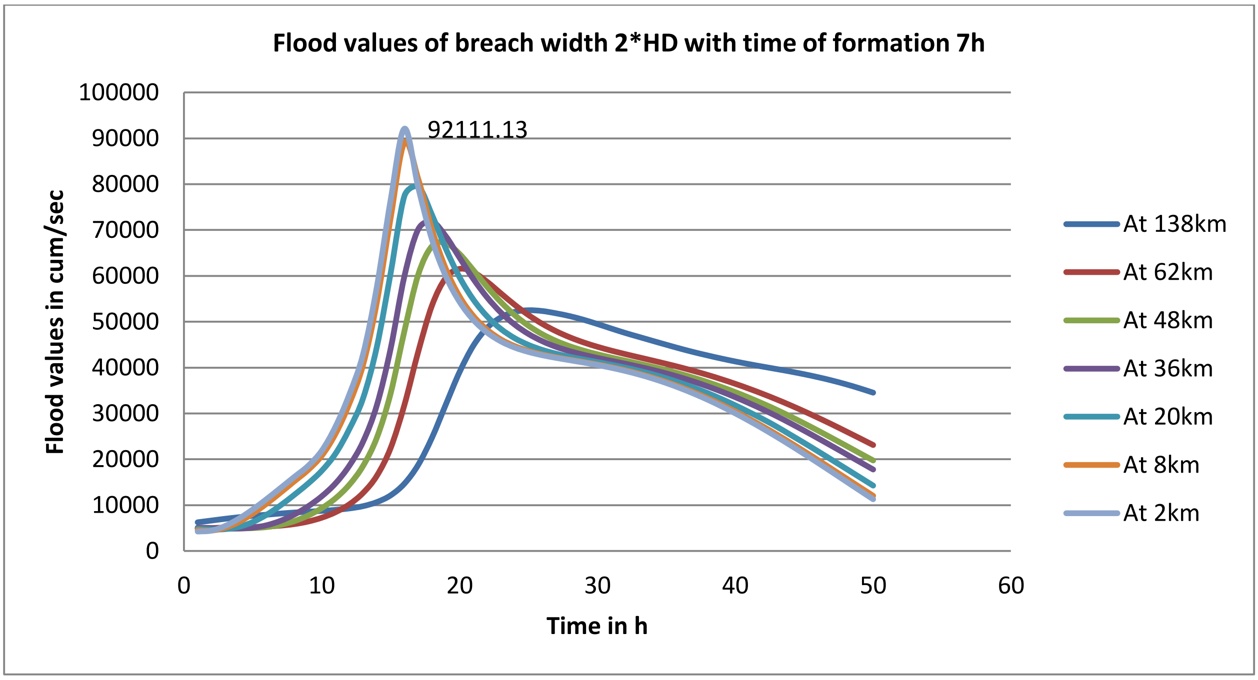


Figure 5.14 Flood values at various stations with breach width 2*HD and breach time 7h

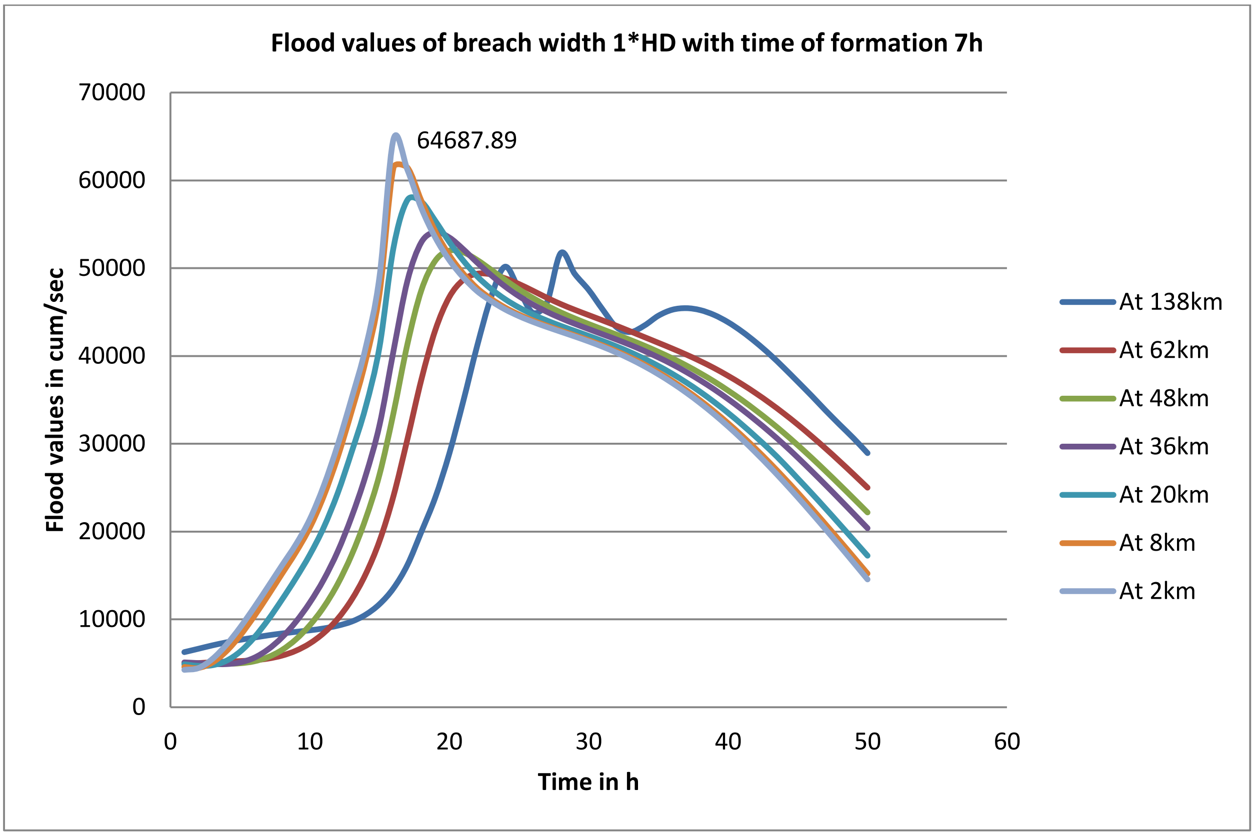


Figure 5.15 Flood values at various stations with breach width 1*HD and breach time 7h

Maximum flood value at 2km location is 64687 m^3/sec. in case of breach width 1*HD and breach time is 1h then flood occurred 68739 m^3/sec. so breach width 1*HD and breach time 1h is low critical.

Water elevation:

Table 5.12 water elevation at station 48km with different breach width and breach time

Breach width	7h	5h	3h	1h
1074	99.91	101.39	104.15	107.55
4*HD	99.89	101.32	103.85	106.33
3*HD	99.79	101.22	102.96	104.53
2*HD	98.48	99.25	99.99	100.32
1*HD	95.39	95.62	95.73	95.85

From the above the table at particular breach time of 7h if breach width is decrease 1074 to 4*HD, 4*HD to 3*HD, 3*HD to 2*HD and 2*HD to 1*HD water elevation decreases with respect to breach width.

Water elevation at same time of formation (7h) and different breach width at various stations:

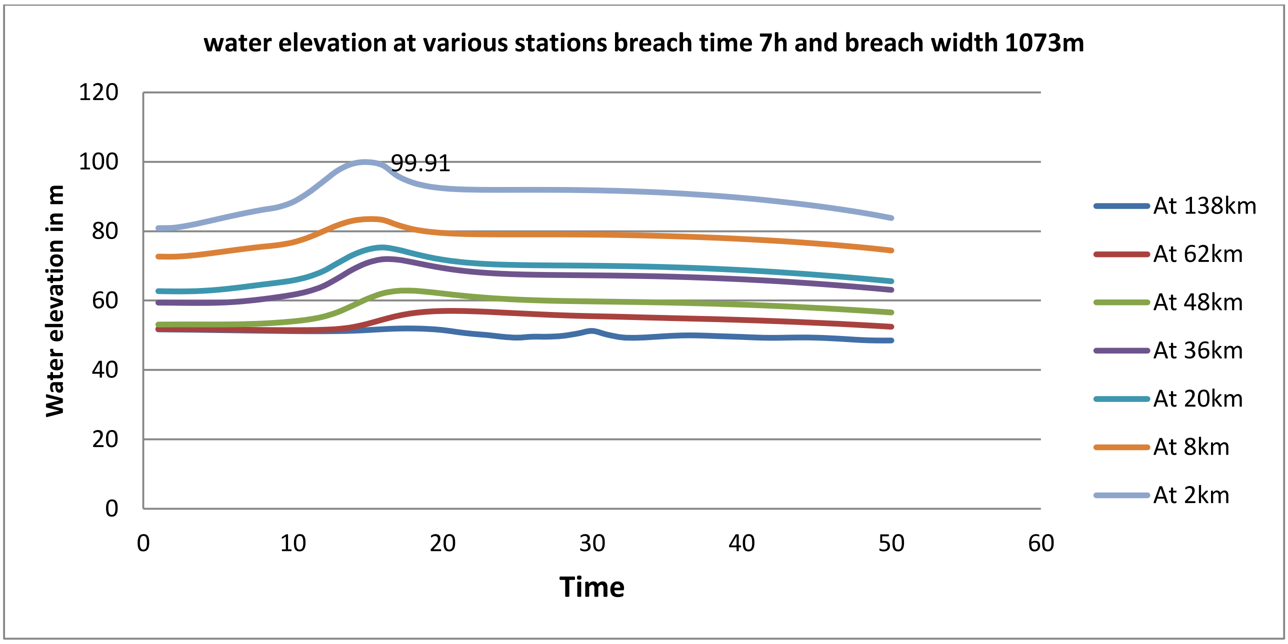


Figure 5.15: Water elevation at various stations with breach width 1074 and breach time 7h

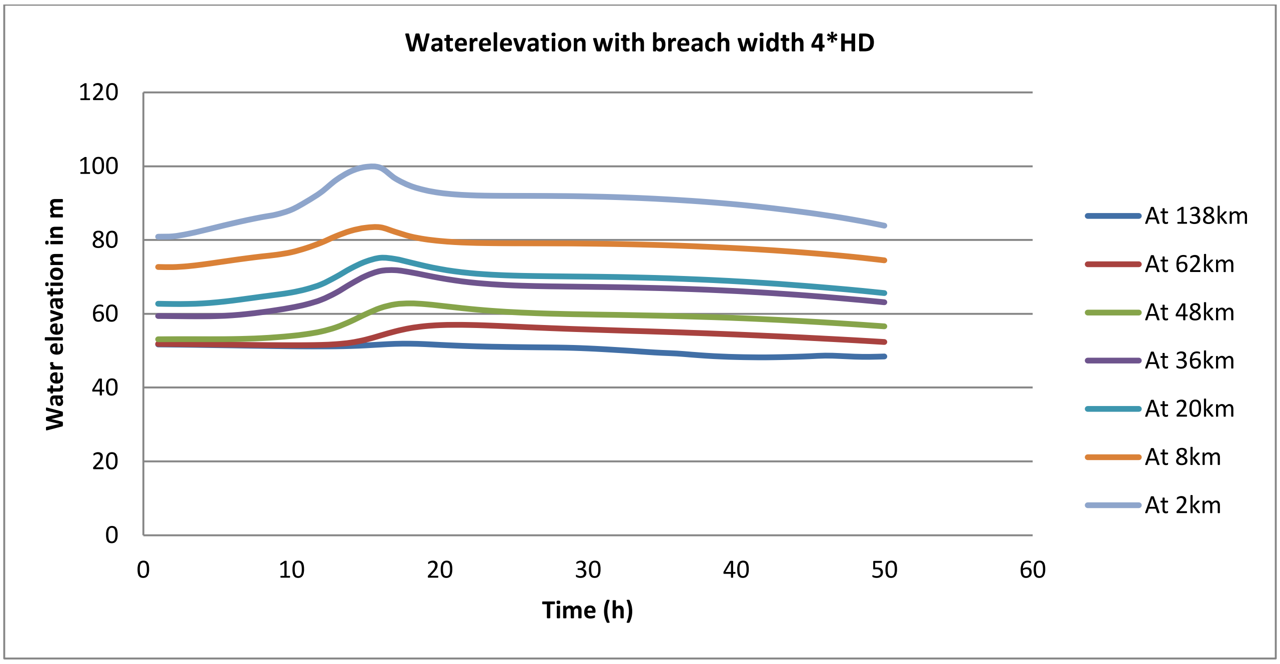


Figure 5.16 Water elevation at various stations with breach width 4*HD and breach time 7h

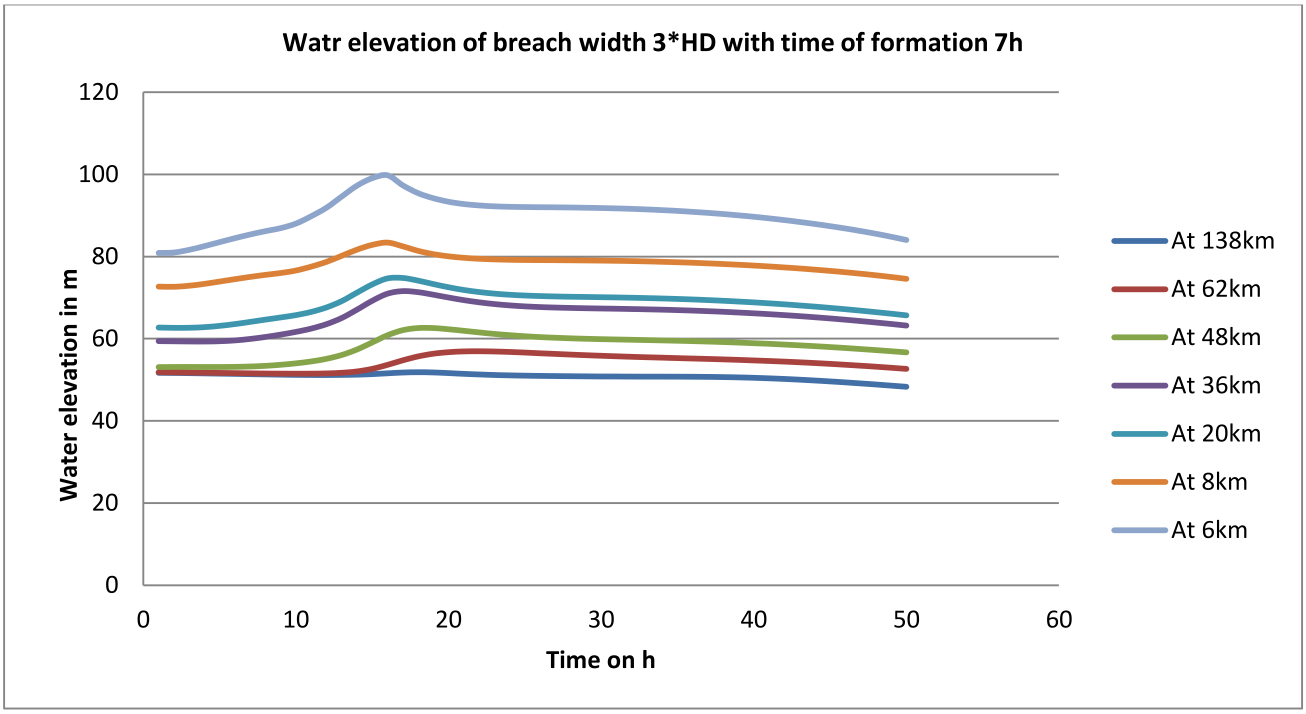


Figure 5.17 Water elevation at various stations with breach width 3*HD and breach time 7h

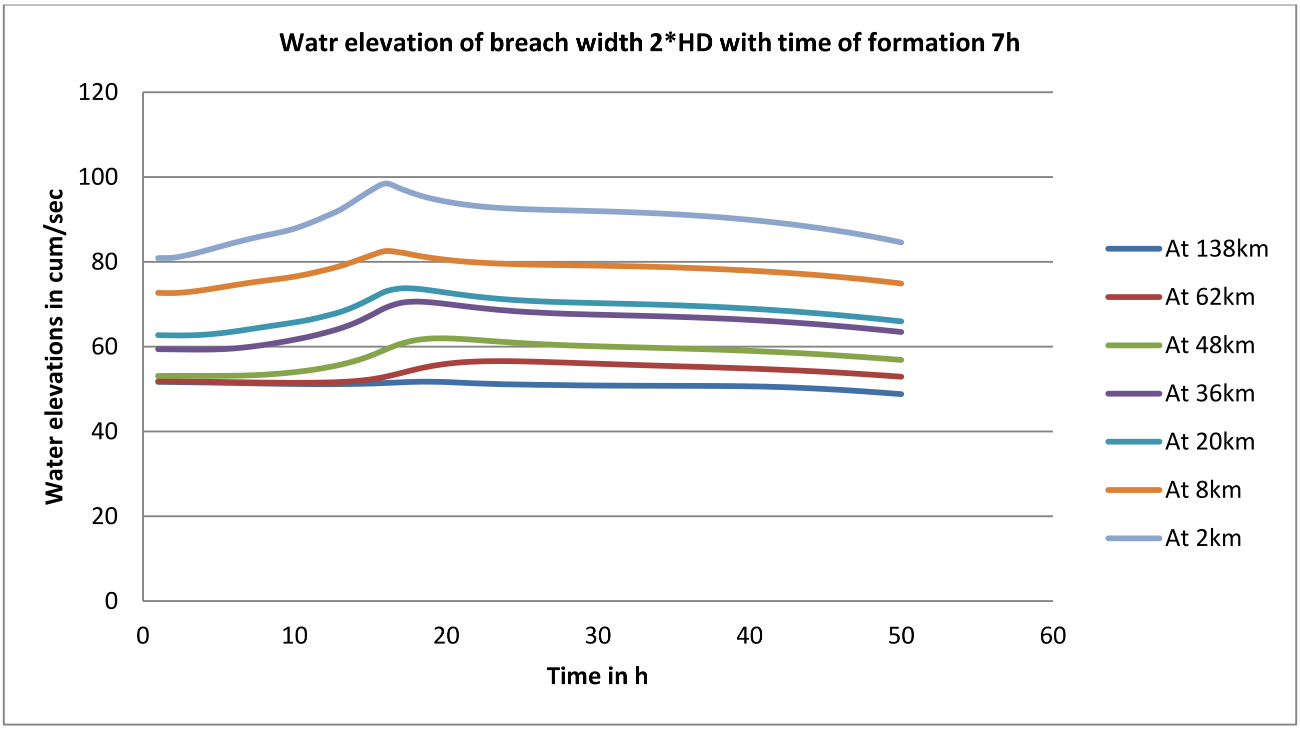


Figure 5.18 Water elevation at various stations with breach width 2*HD and breach time 7h

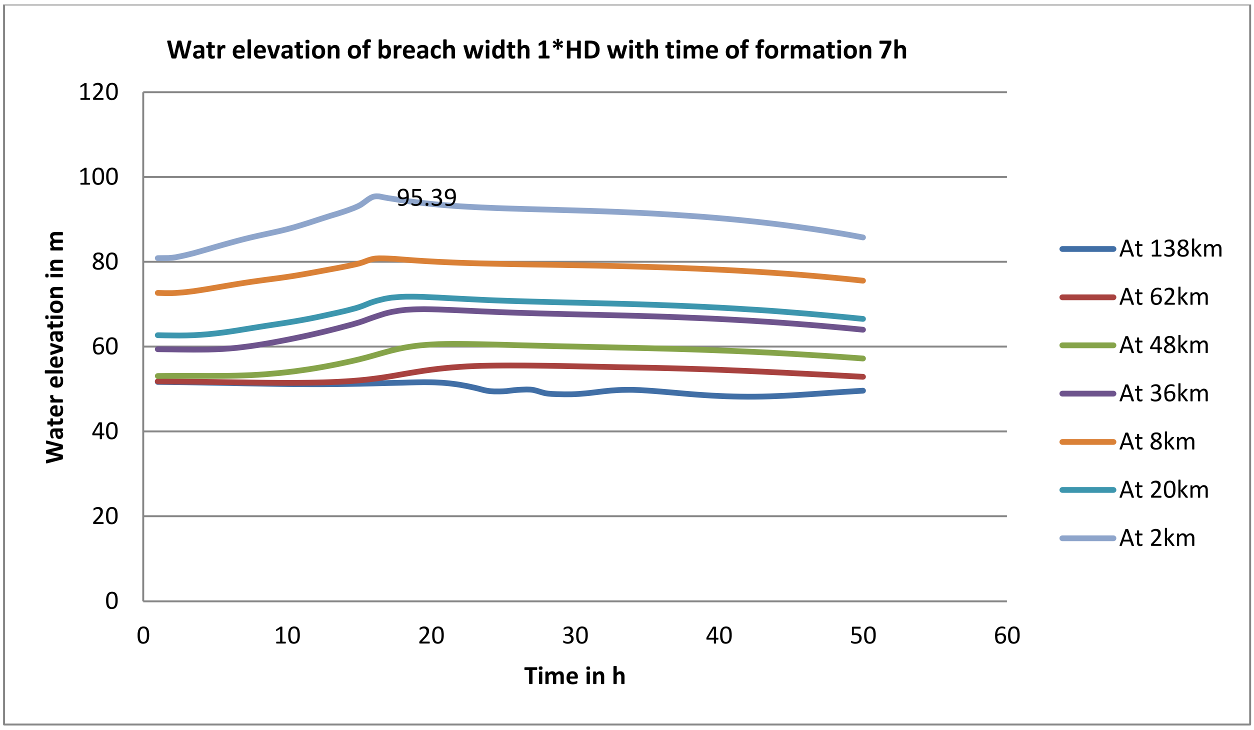


Figure 5.19 Water elevation at various stations with breach width 1*HD and breach time 7h

5.3.3 Effect of Manning's value (k)

Flood values:

Flood values various stations of same breach time 7h and changing manning's values and breach width:

At station 2km from dam

Table 5.13 Flood values at station 2km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	105161	100369	97902
4*HD	104312	100144	97054
3*HD	104046	100001	97019
2*HD	92111	91672	90714

1*HD	64687	63623	62568

Flood value decreases with manning's value .if manning's value is to be constant then breach width decreases then flood values decreases. Maximum flood at k=0.035 is 105161 m^3/sec then it is as decrease as maximum flood at k=0.048 is 100369 m^3/sec and at maximum flood at k= 0.055 is 97902 m^3/sec. it is decreases with manning's values.

At station 8km from dam

Table 5.14 Flood values at station 8km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	104297	99725	97267
4*HD	101678	99008	96850
3*HD	100721	98913	96694
2*HD	890781	88292	87086
1*HD	61462	611953	61360

At station 20km from dam

Table 5.15 Flood values at station 20km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	97417	89803	86669
4*HD	96870	87142	86234
3*HD	86981	86016	86177
2*HD	79486	76460	74908
1*HD	55385	54402	53812

At station 36km from dam

Table 5.16 Flood values at station 36km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	87705	79991	76038
4*HD	84679	78925	75845
3*HD	83139	77125	74032
2*HD	71835	68178	66191
1*HD	54112	53212	51531

At station 48km from dam

Table 5.17 Flood values at station 48km with different breach width and manning's value.

Breach width	K=0.035	K=0.048	K=0.055
1074	80148	72067	67867
4*HD	76753	70870	67786
3*HD	76093	69784	66434
2*HD	64768	62981	60712
1*HD	50936	50118	49146

As we Know, when the Manning's Roughness Coefficient (N) increases there is loss of energy which will affect the wave speed. This loss of energy is dissipated in the atmosphere through the bounding walls of the channel or the water surface. Chow, 1959 has been suggested us the value of Manning's N in the range of 0.03 to 0.055 for the regions showing gravels, cobbles and few boulders at the bottom with no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage as discussed earlier. As expected the velocities reduce with increase in Manning's N, and vice versa. This will affect the maximum water elevation and discharge value also. From observation of above table if increases the manning's value then it's maximum flood value is to be decreases.

Flood values at various stations of different manning's values and same breach width 1074m and same breach time 7h:

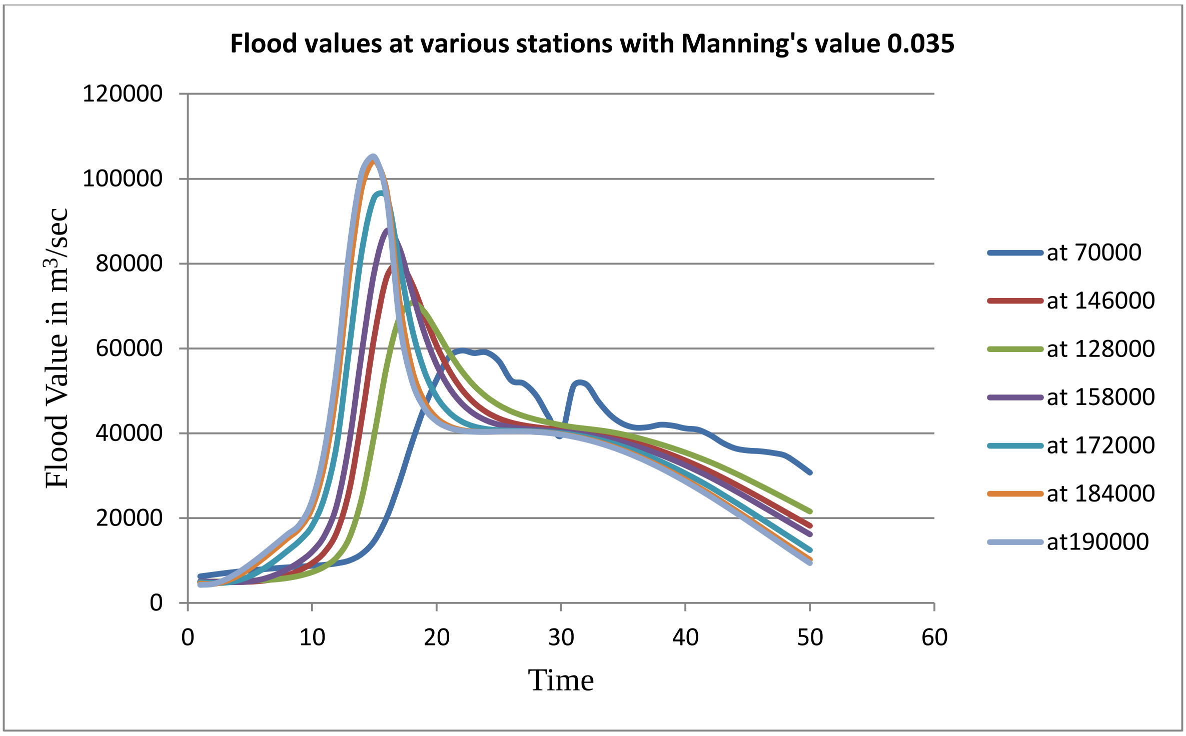


Figure 5.20 Flood values at various stations with manning’s value 0.035

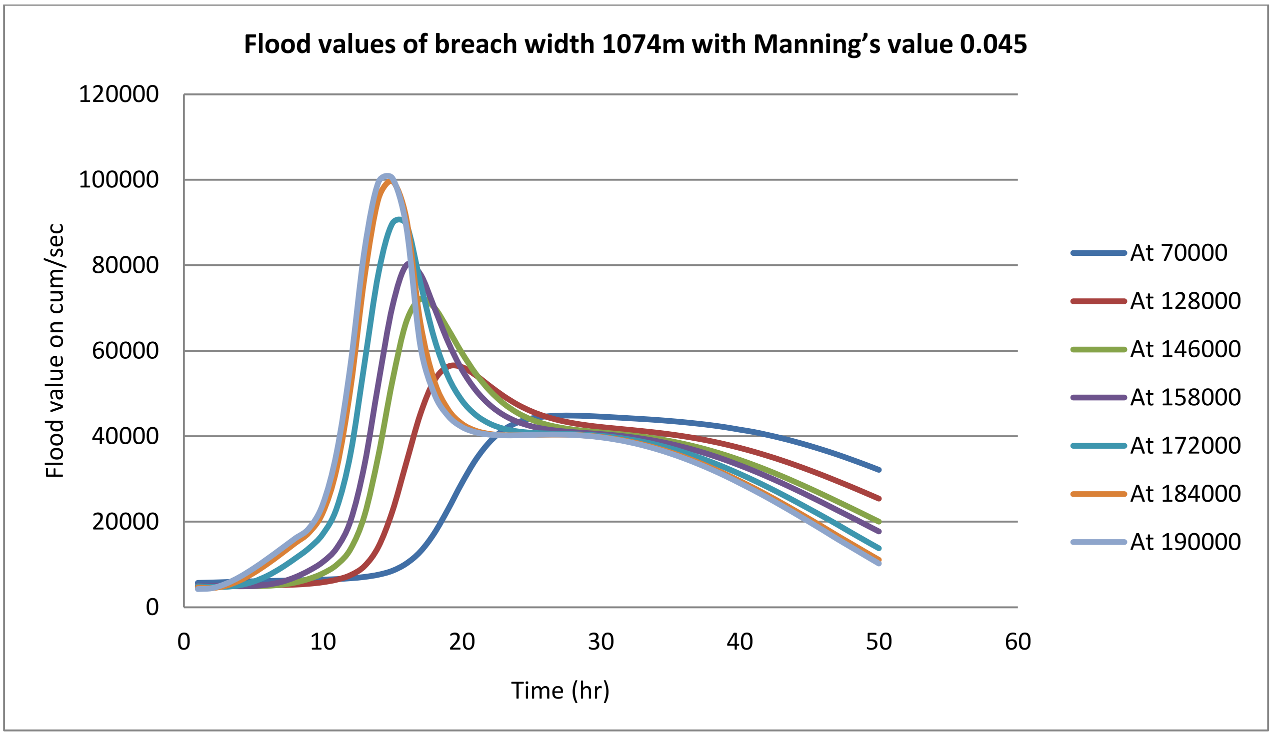


Figure 5.21 Flood values at various stations with manning’s value 0.048

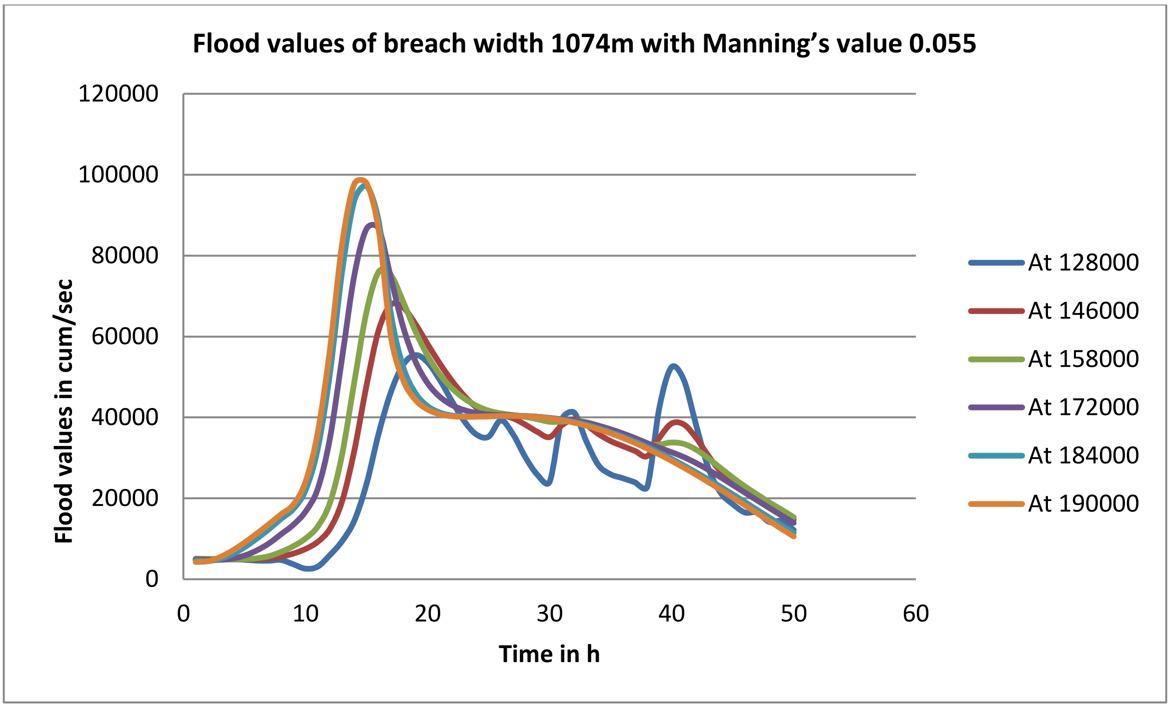


Figure 5.22 Flood values at various stations with manning's value 0.055

Water elevation:

Water elevation various stations of same breach time 7h and changing manning's values and breach width:

At station 2km from dam

Table 5.18 Water elevations at station 2km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	99.71	101.15	101.95
4*HD	99.79	101.19	101.83
3*HD	99.79	101.1	101.73
2*HD	98.49	100.17	101
1*HD	95.39	97.17	97.95

Water elevations increases with manning's value. But flood value is invers processes. Compression Flood value at setup breach width 1074 and manning's value k=0.035 it is 105161 m^3/sec and at which water elevation is 99.71m but at setup of breach width 1074 and manning's value k=0.055 flood value is 97902 m^3/sec at which water elevation 101.95m it is greater than pervious setup.

At station 8km from dam

Table 5.19 Water elevations at station 8km with different breach width and manning's value.

Breach width	K=0.035	K=0.048	K=0.055
1074	83.49	85.72	86.85
4*HD	83.32	85.58	86.81
3*HD	83.42	85.17	86.22
2*HD	82.54	84.8	85.72
1*HD	80.8	82.67	83.5

At station 20km from dam

Table 5.20 Water elevations at station 20km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055
1074	75.32	76.37	77.8
4*HD	75.17	76.3	72.67
3*HD	74.67	76.44	77.16
2*HD	73.73	75.55	76.33
1*HD	71.45	73.53	74.31

At station 36km from dam

Table 5.21 Water elevations at station 36km with different breach width and manning's value

Breach width	K=0.035	K=0.048	K=0.055

1074	71.91	73.23	73.89
4*HD	71.82	73.11	73.68
3*HD	71.59	72.85	73.38
2*HD	70.63	72.02	72.63
1*HD	68.83	70.25	71.04

At station 48km from dam

Table 5.22 Water elevations at station 48km with different breach width and manning's value.

Breach width	K=0.035	K=0.048	K=0.055
1074	62.8	63.96	64.48
4*HD	62.57	63.93	64.36
3*HD	62.6	63.63	64.02
2*HD	61.92	63.09	63.54
1*HD	60.52	61.95	62.7

Water elevation increases with respect to increasing of manning's value but maximum flood value is to be decreases. If manning's value increases then the roughness of the channel increases. So it obstruct the water flow then water elevation is to be increases. Manning's value changes k=0.035 to k=0.048 then flood changes 105161 m^3/sec to 100369 m^3/sec so it is decreases but water elevation changes 62.8m to 63.96m it is increasing value. So manning's value important criteria for dam break analysis.

Water elevation at various stations of different manning's values and same breach width 1074m and same breach time 7h:

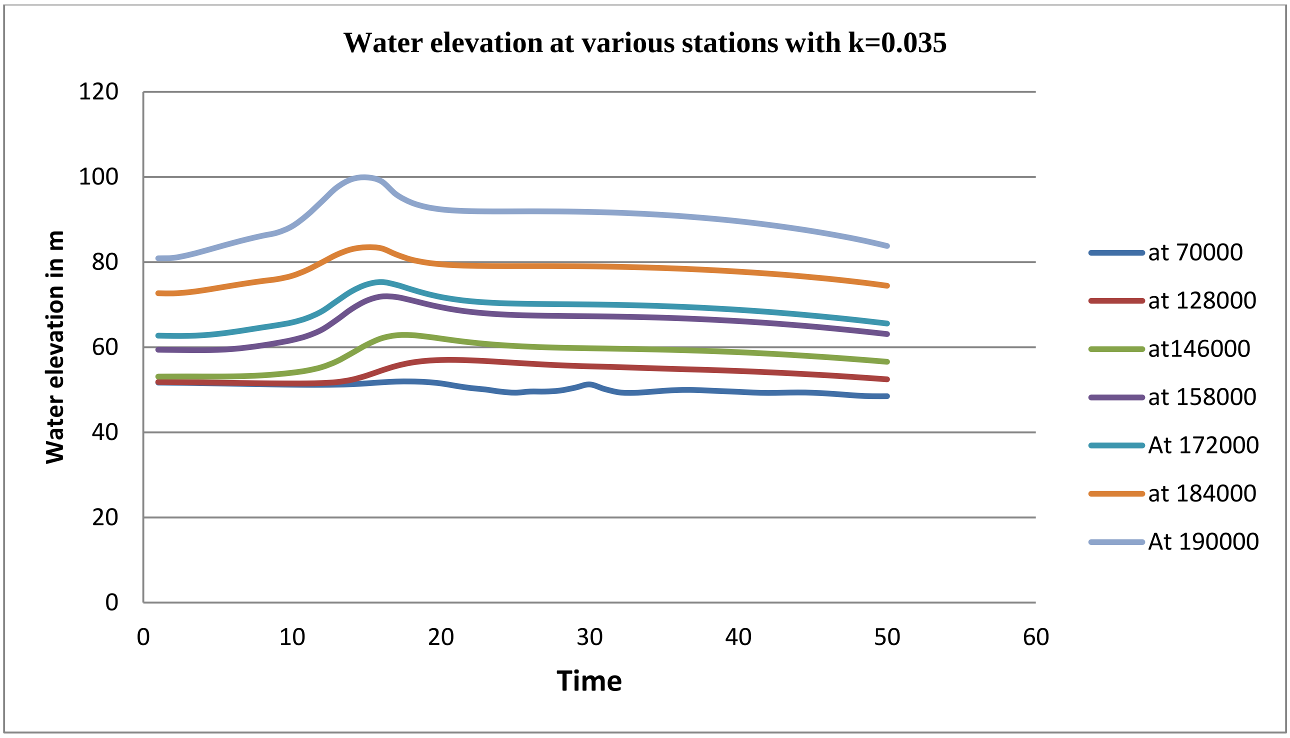


Figure 5.23 Water elevation at various stations with manning's value 0.035

Manning's value changes k=0.035 to k=0.048 then flood changes 105161 m^3/sec to 100369 m^3/sec so it is decreases but water elevation changes 62.8m to 63.96m it is increasing value. So manning's value important criteria for dam break analysis.

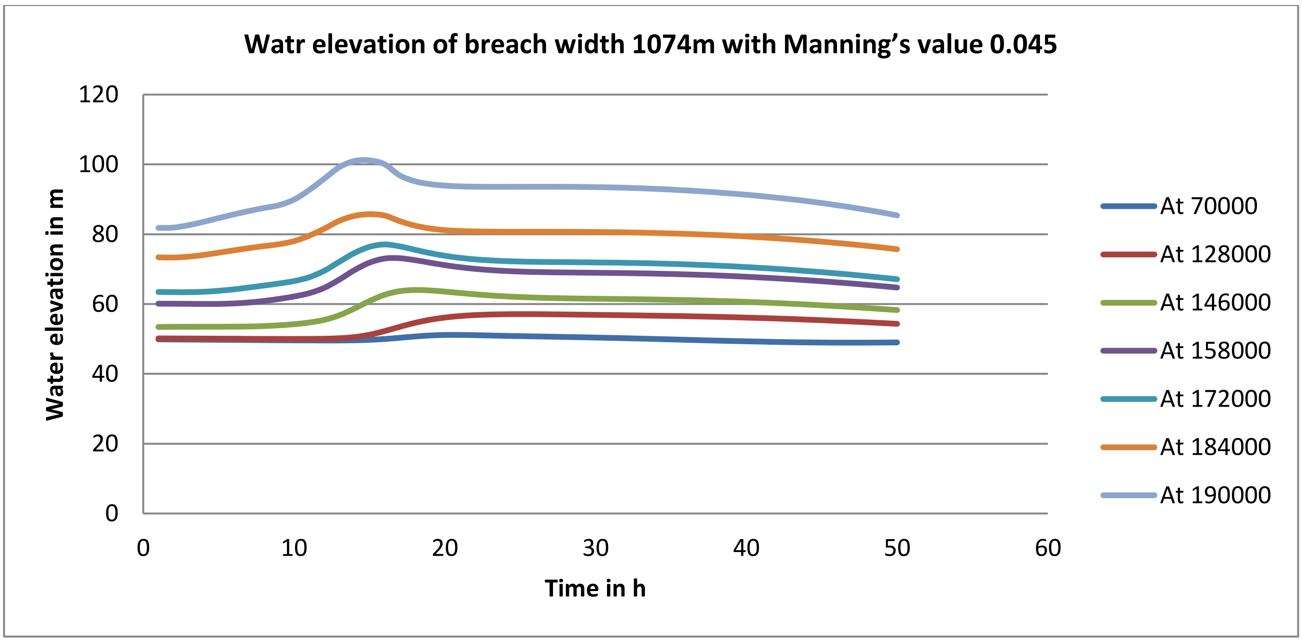


Figure 5.24 Water elevation at various stations with manning's value 0.045

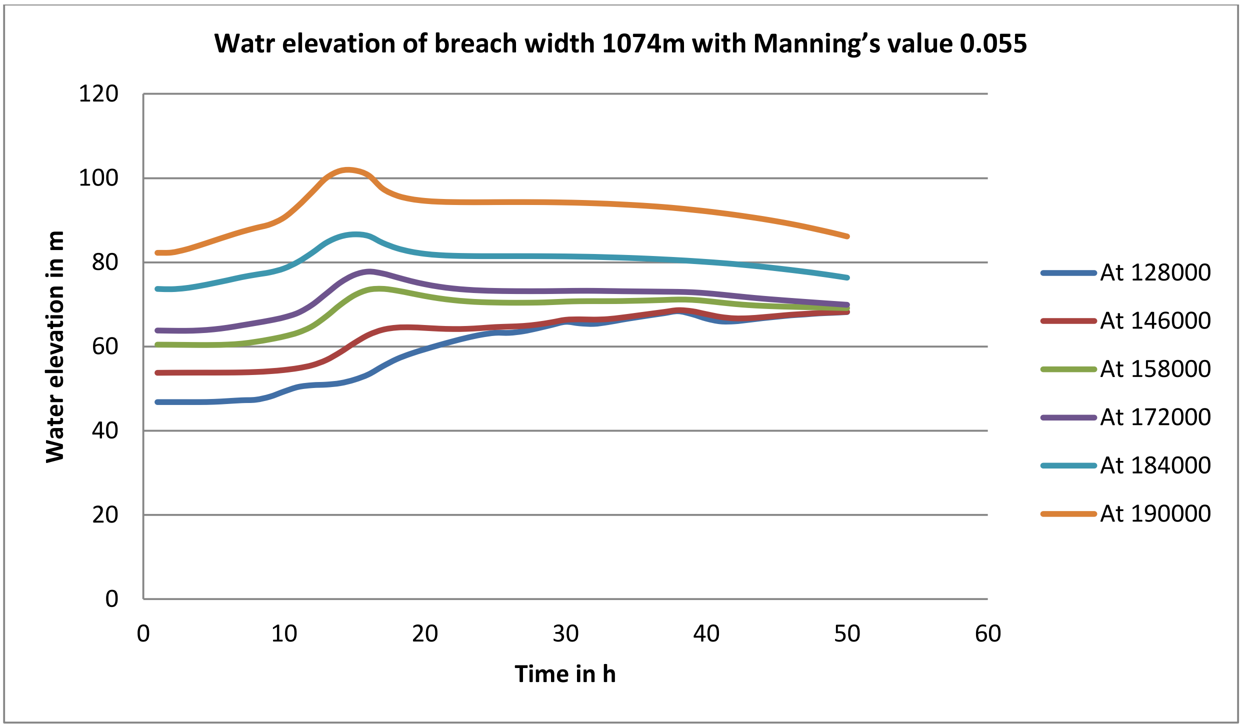


Figure 5.25 Water elevation at various stations with manning's value 0.055

5.4 Comparisons of Flood values and water elevations in without failure and with failure:

5.4.1 Comparison of flood values:

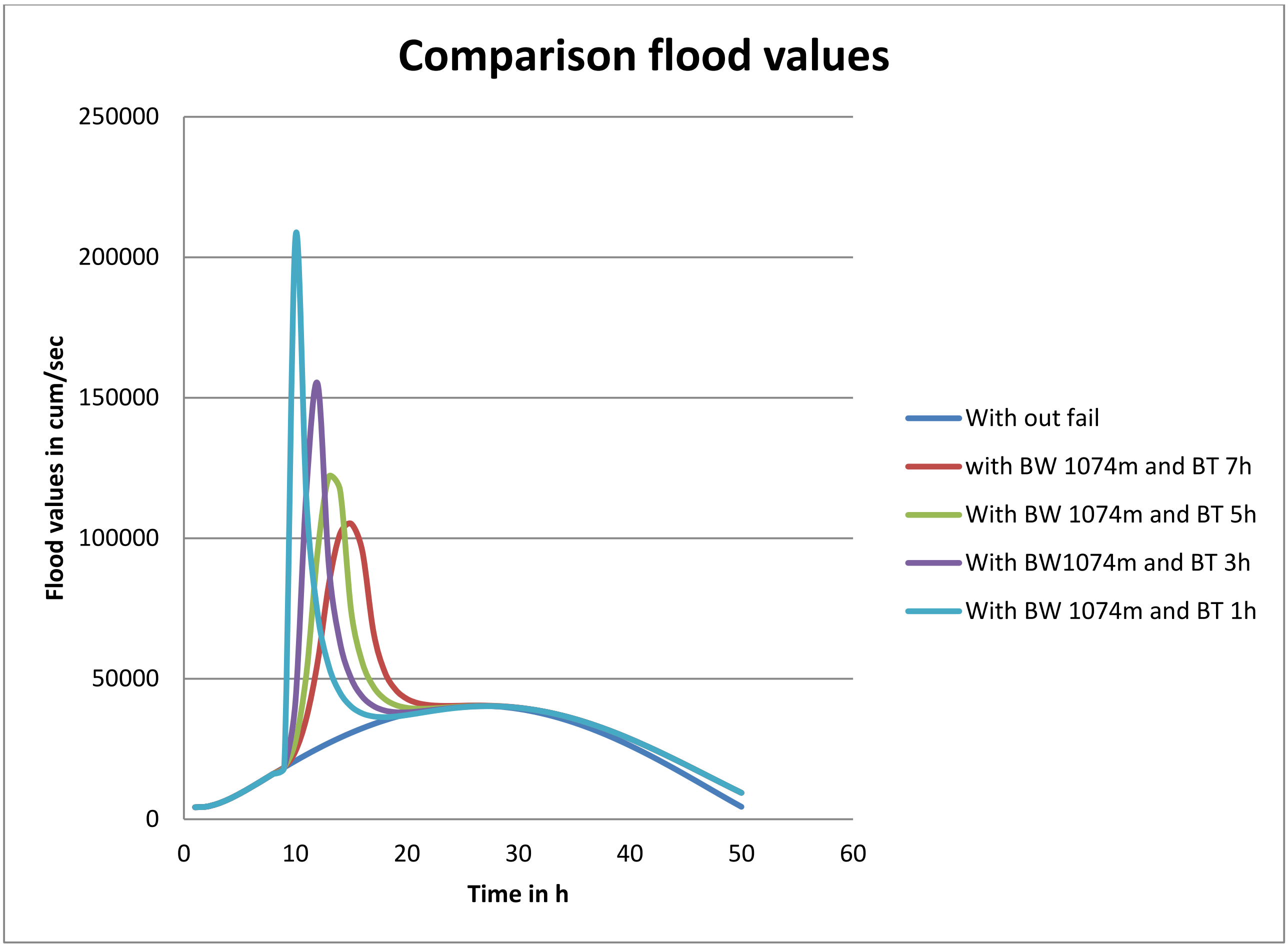


Figure 5.26 comparison of flood values in without failure and with failure at location 2km from dam

Maximum flood occurred in without failure of dam is 40430 m^3/sec. it is compare with setup of breach width 1074m and breach formation time 7h in which maximum flood occurred 105161 m^3/sec. it is 2.6 times more than flood value of without failure of dam. In another setup of breach width 1074km and breach time 5h flood value is 121946 m^3/sec. it is 3.1 times greater than the flood value without failure of dam and another setup of breach width 1074m and breach time 3h is to be 154853 m^3/sec which is 3.85 times greater than the flood value without failure of dam. In one more setup of breach width 1074m and breach time 1h of flood value is to be 206953 m^3/sec which is 5.1 times greater than the flood value without failure of dam. The setup of breach width 1074 and breach time 1h is critical.

5.4.2Comparison of water elevations

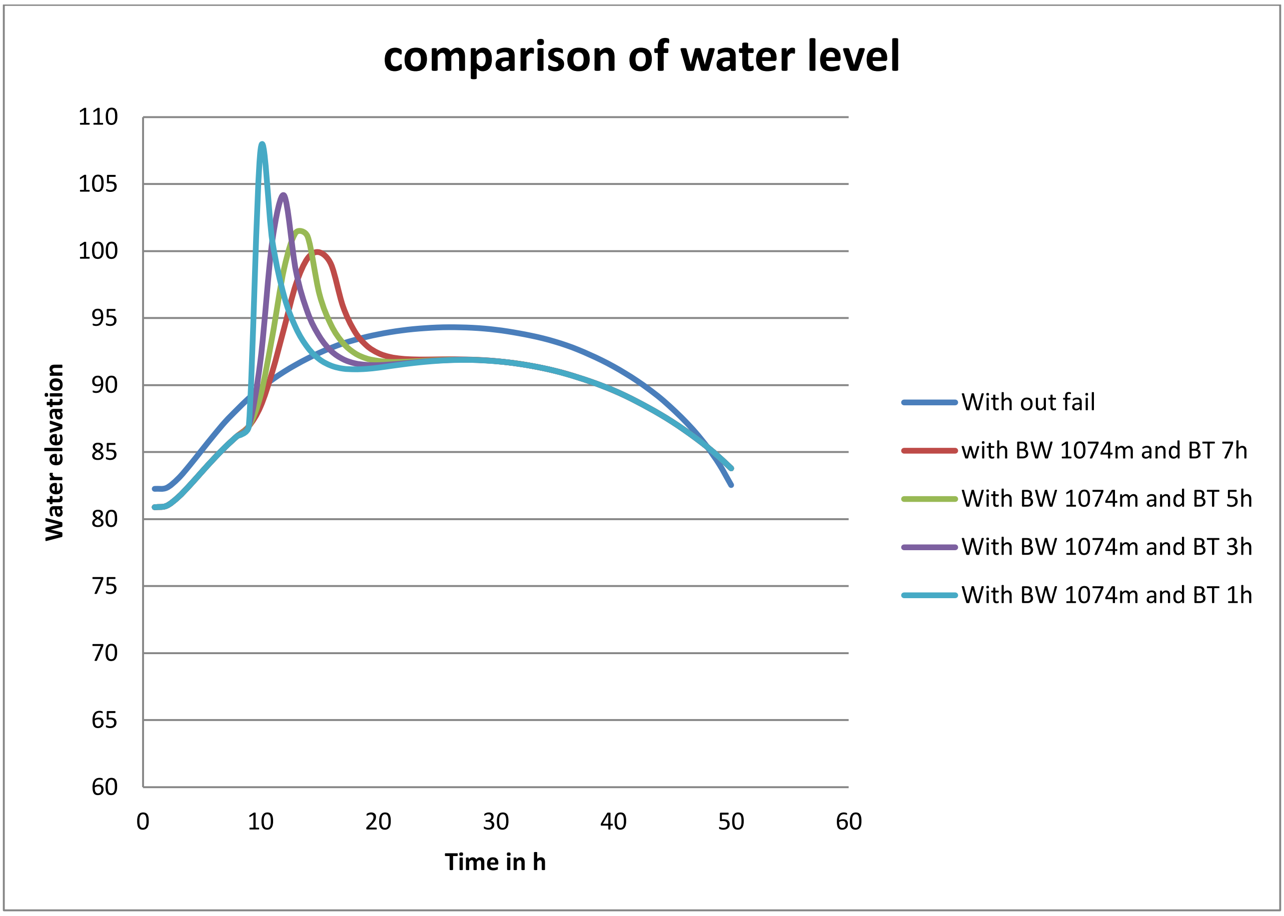


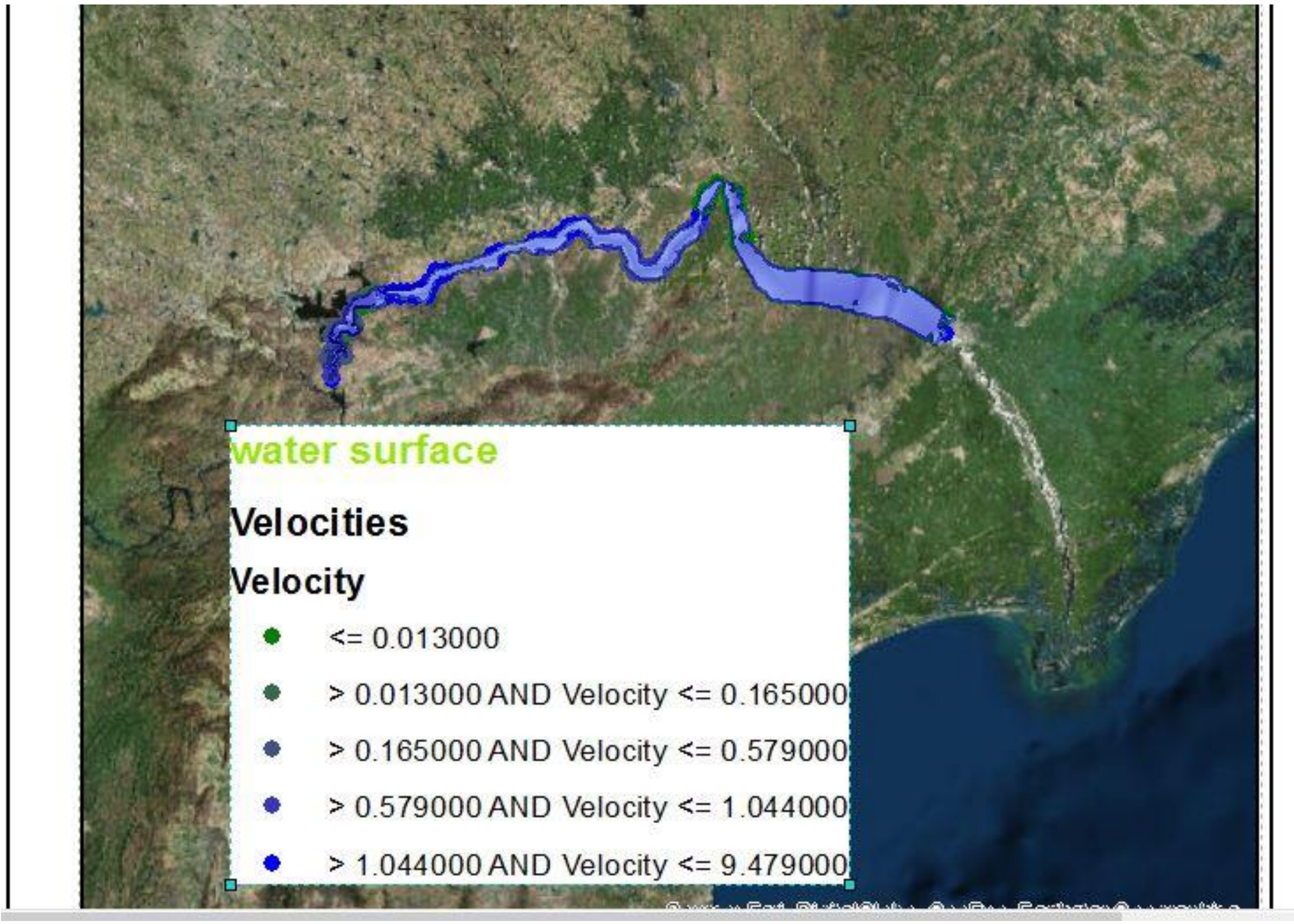
Figure 5.27 comparison of Water elevations in without failure and with failure at location 2km from dam

Water elevation at 2km of without dam failure it is to be 94.15 but compare with reaming setup water elevations are 99.71, 101.39, 104.15, 107.55 respectively.

5.5 Flood inundation map:

Dam breach flood inundation map indicates areas that may be flooded as a result of a dam failure. Basically an inundation map depicts isolines of flood depth downstream of the valley. The maps would be used by wide range of end-users for planning and as a response tool to determine the effects of dam failure in downstream areas. For this study, flood inundation maps were generated using HEC-GeoRAS. GIS information was exported from HEC-RAS and read into GIS with GeoRAS. The geo-referenced cross sections imported and water surface elevations attached to the cross-sections was used to create a continuous water surface. The water surface was then compared with the terrain model and floodplain is identified where the water surface is higher than the terrain. This inundation map was used to carry out disaster management plan for the inundated area.

Flood map for breach width 1074m and breach formation time 7h:



In flood maps creates water surface velocity of reach length of 144km. The velocity is maximum at near to downstream of dam, it is 9.47m/sec. This velocity occurred at 2km to 48km of downstream of dam. The velocities decreased near to last downstream station.

Chapter 6

Conclusions and Recommendations

In this thesis the simulation of hypothetical failure of "Nagarjunasagar dam" is carried out, this is earth fill dam having height of 124m. The impact of Dam Break in the downstream area is observed in terms of flood hydrograph, flood duration, water elevation and flood map. Further the sensitivity analysis of Breach Time, Breach Width, and Manning's Roughness is carried out. Conclusions are drawn by comparing their results as written bellow.

- In our dam break analysis the Peak discharge is 105161 m^3/s which is 2.8 times greater than the probable maximum flood and in another critical setup of breach width 1074m and breach time 1h then we got peck flood value 206988 m^3/sec which is 4.9 times greater than the PMF.
- As from the sensitivity analysis of the dams we conclude that effect of breach time on discharge is much more pronounced than the water elevation.
- If breach formation time decreases then flood values increases gradually. So in breach analysis breach formation time is important criteria.
- Form our observation of flood values and water elevations stations 2km, 8km, 20km form dam occurred maximum floods.
- The flood values are decreases if manning's values are decreased but water elevation is to be increases because of roughness increases at that cross section.
- Setup of breach width 1074m and breach time 1h is critical setup in which maximum flood occurred it is to be 206988 m^3/sec it is 5 times more than our PMF. So it is critical setup all over setups of breach width and breach time.
- Comparison of flood values of without failure of dam 40430m^3/sec and our critical setup of breach width 1074m and time of formation time 1h of flood value 206933 m^3/sec which is 5.1 times more.

- Comparison of water elevation of without failure 94.15 m it is to be our critical setup of breach width 1074m and time of formation time 1h is 107.55m water elevation gradually increased 13.4m of water levels.
- From my thesis concludes stations 2km, 8km, 20km from dam are critical locations. Up to 36km from dam flood values occurred more than PMF value.
- Maximum velocities occurred at 2km to 48km of downstream side. So location between 2km to 48km is critical.
- HECRAS is one dimensional software. It gives the water elevation with respect to datum, but did not give result of depth of water in river. So this can processed with 2D modeling software's as ANSYS software.
- Scope of future work is upstream dam of Nagarjunasagar is Srisailam dam if it is failed then find out the flood routing over in Nagarjunasagar reservoir. Then find flood hydrograph at downstream side of Nagarjunasagar dam and flood hydrograph of in between Srisailam dam to Nagarjunasagar.

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